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AN EXCHANGE OF IDEAS
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FRACTURE CRITICAL BRIDGES: WHAT DISTRICTS NEED TO KNOW

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FIGURE 1: Mianus River Bridge, 1983.
(Courtesy of Dr. John Fisher, Lehigh University.)

INTRODUCTION

America's bridges and highways are aging and are required to carry more vehicles, in many cases, than their designers ever envisioned. A national tendency in years past has been to postpone routine maintenance and to divert the funding elsewhere when money was in short supply. This tendency to postpone maintenance has speeded the deterioration of these structures and pavements. Bridges are a primary concern, particularly in light of the number of bridge failures in the last ten years.

Unlike many serious pavement problems, the collapse of a bridge is an obvious, dramatic event (Fig. 1). Everyone notices. Even if no lives are lost in the catastrophic failure of a bridge, the replacement cost of the structure is enormous, often running millions of dollars, usually more money than it would have cost to do routine inspections and maintenance over the years. In response to this problem, more emphasis is now being put on inspection and maintenance.

INSIDE...

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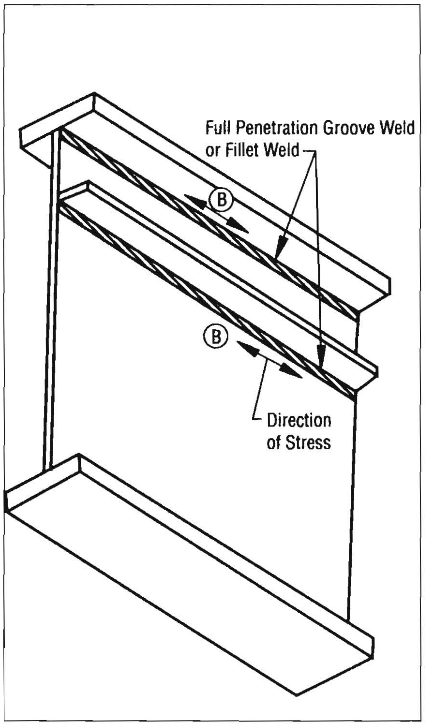

FIGURE 2: Direction of stress in Category B web-to-flange connections and longitudinal stiffener [Ref. 6].

Fatigue is the occurrence of material failure by repeated applications of loads which, when induced once or infrequently, would not have an effect on the material. Fatigue cracking is one of the two main causes of catastrophic bridge failure, scouring being the other one. This article will deal with the identification and inspection of fracture critical bridges for fatigue cracking.

The Federal Highway Administration has mandated that all of Texas' fracture critical bridges must be inspected by March 1992 or the department may face sanctions. District inspection personnel need to know what constitutes a fracture critical bridge so that they can identify it as such in the BRINSAP File and trigger the appropriate special inspection [Ref. 1]. Approximately 350 fracture critical bridges have been identified so far, and of these, over 50 percent have been inspected.

WHAT ARE FRACTURE CRITICAL BRIDGES

While all bridges have an estimated fatigue life and may develop fatigue cracking, not all bridges are classified as "fracture critical." A simple definition of a fracture critical bridge is any bridge where failure in a tension area of a ***steel*** member would cause immediate collapse of the span. For the moment, fracture can be thought of as instantaneous crack growth. Critical fractures always occur in tension areas and perpendicular to the direction of stress (Fig. 2).

Examples of fracture critical types are bridges with two girder steel spans, bridges with transverse steel caps (box or I-beam), with steel floor beams, suspension bridges, and truss bridges. Three girder steel bridges with exceptionally wide spacing between girders are also considered fracture critical.

Fracture Mechanics in Brief

A detailed account of fracture mechanics is outside the scope of this article; however, some explanation needs to be given in order to understand why some fatigue cracking results in fracture and some does not. Fracture mechanics is based on analysis of the state of stress at the tip of a crack in a material [Ref. 2, p.626]. Stress in a material loaded in tension is simply the load on the member divided by the cross-sectional area of that particular part. The shape of the part, therefore, is extremely important. If there are sharp reentrant corners (Fig. 3), notches, holes, partial penetration welds in a detail or defects such as gouges and welding flaws, the stress concentration will be much higher near these features than in the member as a whole. Cracks tend to form at these features under cyclic loading. Stress will concentrate at the tip of the crack, tending to pull the material apart.

Critical crack length is the point where more energy will be released by the material as it is pulled apart (relaxation of strain) than is required to initiate cracking [Ref. 4, p.100-101]. In other words, critical length cracks get longer without outside help.

Critical crack length is different for every material. It is dependent on the inherent fracture resistance of the material, the shape of the part, environmental factors such as ambient temperature, corrosion, and number of loading cycles [Ref. 2, p.626].

Cracks that are shorter than the critical length for a given material are stable and do not normally extend unless loaded again to a stress sufficient to initiate more cracking. Bridges are all too frequently subjected to overloads which can initiate or extend cracks. Each time a crack extends, the closer it gets to its critical length. Cracks that are critical length or longer self-propagate and are very dangerous. As Professor J. E. Gordon points out, "Such cracks spread faster and faster through the material and inevitably lead to an 'explosive', noisy and alarming failure. The structure will end with a bang, not a whimper, and very possibly with a funeral" [Ref. 4, p.102].

HOW ARE FRACTURE CRITICAL BRIDGES PRIORITIZED

Districts with numerous bridges are likely to have more than one frac-

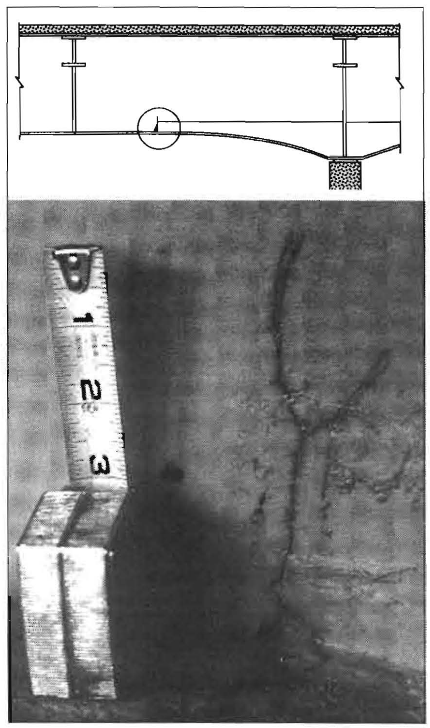

FIGURE 3: Cracking at sharp corner of welded insert [Ref. 6].

ture critical bridge. Which one should be inspected first? As with most things in life, it helps to have a set of priorities. For fracture critical bridges these priorities are:

1. **Age**. The older the bridge, the more repeated loads, weathering, overloads, etc., it has been subjected to. The volume and variation of live load traffic on the bridge may be radically more than the bridge's designers allowed for, which would shorten the bridge's useful life.

Older bridges often have more corrosion damage (Fig. 4), as well as being nearer the end of their estimated fatigue life. Corrosion can separate details, changing their alignment. It also speeds up the rate of cracking by decreasing the cross-sectional area of a corroding member, which increases the applied tensile stress to that member [Ref. 3, p.116].

A consideration for welded bridges is that in the 1940s and 50s nondestructive testing techniques for inspection of weldments were not as effective as those now available in detecting flaws, so older welded bridges may have more uncorrected, hidden weld problems than new bridges [Ref. 6, p.50].

2. **Stress Range**. The magnitude of live load and impact stresses (the stress range), along with the number of stress cycles these loadings cause, are primary factors affecting the fatigue life of a bridge. Fatigue life is inversely proportional to stress range, except at very low magnitudes. (Low magnitude stress cycles can be repeated indefinitely without causing fatigue.) Ideally, bridge parts are designed so that the actual range of stress does not exceed the allowable fatigue stress range outlined in Section 10.3 of the ***AASHTO Standard Specification for Highway Bridges*** [Ref. 5, p.112-117]; however, conditions in the field are often far from ideal. Impact damage and/or settling can cause out-of-plane deformation of structural members. Out-of-plane bending can result in very high magnitude stress ranges — far beyond what was accounted for by designers (Fig. 5). These very high magnitude stress ranges can cause fatigue damage. Fracture critical bridges with out-of-plane bending problems or higher-than-designed-for truck traffic should take priority over similar age fracture critical bridges without these factors.

3. **Nature of the Fracture Critical Detail**. AASHTO has categorized common steel bridge structural details according to their fatigue resistance, A through E′, with A being the most fatigue resistant and E′ being the least (See ***AASHTO Standard Specification for Highway Bridges***, Table 10.3.1B). Chapter 2: "AASHTO Categories of Fatigue Strengths," in ***Manual for Inspecting Bridges for Fatigue Damage Conditions*** [Ref. 6], is an excellent reference. If two bridges of similar ages and stress ranges are being scheduled for fracture critical inspection, the one with worst fatigue strength details should be given priority.

4. **Span Type (Simple or Continuous).** All else being equal, a simple span is more at risk of fracture failure than a continuous span.

5. **Internal Redundancy**. All bolted or riveted components are generally less prone to fracture than welded members (Fig. 6). Cracks that might be stopped by a riveted or bolted splice propagate through the material continuity of a weld.

6. **Damaged Members**. Any fracture critical component damaged by impact, yielded by overload or otherwise compromised by any unexpected

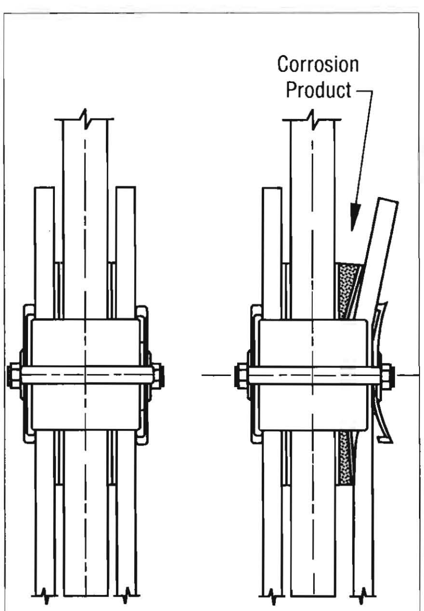

FIGURE 4: Corrosion causing significant realignment of details such as hangers produces very high bearing stresses. (Courtesy of Dr. John Fisher, Lehigh University.)

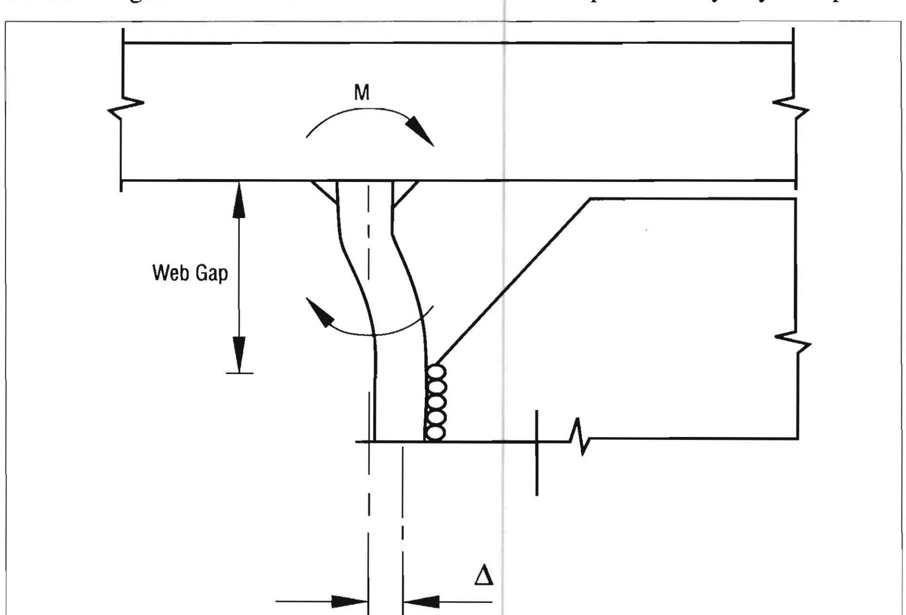

FIGURE 5: End deformation introducing out-of-plane bending in web regions under normal service loads. Web distortion is one of the major sources of fatigue cracking in steel bridges. (Courtesy of Dr. John Fisher, Lehigh University.)

FIGURE 6: Fracture at weld-filled holes [Ref. 6].

detrimental occurrences (such as fires) should be thoroughly inspected as needed.

FRACTURE CRITICAL INSPECTIONS

How to Trigger a Fracture Critical Inspection

When a bridge is recognized as being fracture critical, district personnel must update the BRINSAP File record for that bridge. Item 88B, "Special Flags, Number of Fracture Critical Areas," on card 7 is the specific item that will need to be updated [Ref. 1, p.60]. Fracture critical areas are those places within a fracture critical member where fatigue cracking is likely to begin. For example:

- **Two-girder system** — each girder will have a fracture critical area at the lower flange and lower web areas in the midspan region and at the upper flange and upper web areas in the region over each continuous support. Floor beams can also be fracture critical. A typical poor detail encountered is lateral bracing connected to horizontal gusset plates which are welded to webs in tension regions. The end of the gusset plate is a highly fatigue prone area (particularly when the gusset plate is not attached to the floor beam [Fig. 7] or diaphragm connection plate). A vertical crack here would grow toward the tension flange and might result in fracture of the flange.
- **Two-girder system with suspended span** — in addition to the fracture critical areas found in all two-girder units, the pin and link details at the ends of the suspended span are fracture critical areas. Corrosion that could restrict free movement of the detail and wear on the pins should be closely evaluated. Ultrasonic testing of these pins will be scheduled by the Bridge Division at a future date.
- **Box girder system (two girders)** — As in all two-girder units, all the tension regions of the flange plates and web plates (and possibly the floor beams) are fracture critical areas. The girders must be inspected closely on their exterior and, where possible, on their interior. Typically, the interior of box girders have the worst fatigue prone details. The Bridge Division will contact respective districts prior to fracture critical inspection so access into the girders can be provided.
- **Tied arches** — The hangers and the tie girders of tied arch bridges are fracture critical. Floor beam connections to the tie girders are typically areas of poor fatigue details.
- **Steel box/I-beam caps** — as with two-girder systems, all tension regions, lower flanges and webs at midspan and upper flanges and webs over supports

FIGURE 7: Cracking in a girder web at a floor beam connection plate which was not welded to the top flange. (Courtesy of Dr. John Fisher, Lehigh University.)

are fracture critical areas and should be closely inspected. Like box girders, box caps must be inspected in their interior. The Bridge Division will contact the responsible district for access.

- **Two truss systems** — all tension components including bottom chords, tension verticals, and tension diagonals are fracture critical. Floor beams may also be fracture critical.
- **Suspension span systems** — suspension-type spans such as cable-stayed and the more traditional cable suspension or eyebar chain suspension spans are fracture critical. Fatigue prone components include the main cables or eyebar chains and the suspenders. When used, floor beams can be fracture critical as well.

The total number of fracture critical areas on each fracture critical bridge is calculated and entered in BRINSAP File 88B. The number of fracture critical areas gives the BRINSAP team a good idea of how long the inspection will take and how often the Snooper or bucket truck will have to be moved.

According to BRINSAP data requirements, changes to existing structure data must be made within 90 days of the inspection or evaluation that reveals the change in status of the structure [Ref. 1, p.1]. The files are reviewed by the Bridge Division's BRINSAP Section, who schedule and perform fracture critical, as well as underwater, bridge inspections. Since there is less than a year left to complete the FHWA-required inspections, the district BRINSAP liaison should also call the BRINSAP Section as soon as a fracture critical bridge is identified to make sure the bridge can be scheduled for inspection before March 1992. Remember that bad weather can delay inspections and force them to be rescheduled, so it is best to get the bridge scheduled as early as possible.

What is Needed During an Inspection

To perform a valid fracture critical inspection, the inspector must be no farther than arm's length from the structural members to be inspected, and the bridge must be clean. Repainting is therefore a good time for fracture critical inspection because the structure is down to bare metal and cracks that might be hidden by rust or under paint are easily detected. If a district has a fracture critical bridge which is about to be repainted and is due for a fracture critical inspection, call the BRINSAP Section, and they will work it into their fracture critical inspection schedule. For fracture critical inspection of a bridge not scheduled for repainting, district maintenance forces need to hose the bridge down before the inspection, concentrating on the caps and girder flanges to remove debris and loose rust. Usually, an inspector needs to use the Reachall UB 60 Snooper to get close enough to the structural members. The Equipment and Procurement Division (D-4) requires the transfer from the district of up to $140 per hour for the Snooper and its operator. (The price varies according to how many hours a month the Snooper can be kept in action in a locale.) The BRINSAP Section is responsible for scheduling the Snooper for inspections.

For any inspection that involves the use of the Snooper, traffic control coordination is an important consideration, both for the safety of the traveling public and the people involved in the inspection. The Snooper is wide and, even on a bridge with full-width shoulders, will block part of a lane. On narrow, older bridges, the Snooper

WHY FRACTURE CRITICAL BRIDGES ARE BUILT

Until ten or fifteen years ago, fracture mechanics was a matter for theorists and not well enough known to be applied every day in the design of structures. Consequently, many fifteen-year-old and older bridges have built-in problems. The design shift after the Second World War away from bolting or riveting to the more economical welding of structures adds another factor. Welded bridge details are at more risk of failing from fatigue cracking than are bolted or riveted details. In part, this is because flaws or defects in the welds are areas of high stress concentration — natural starting places for fatigue cracks. Because welding creates material continuity between members, cracks might be stopped by the discontinuity of a bolted or riveted splice propagate through welded connections.

Suspension bridges have been and are used where piers are impractical to place and the span must be carried a long distance. The longest clear span in the world (more than a mile), is a suspension bridge that will open in Denmark in 1993-94. All suspension bridges are fracture critical whether the main cables are multistrand or eyebars. All types of vertical suspenders are considered fracture critical as well.

Truss bridges came into favor in the late nineteenth century because of their combination of stiffness and economy. They remained a much-used design until reinforced concrete started to be used widely for bridges in the 1920s and 1930s. Most trusses are at low risk of catastrophic failure by fracture because they have multiple separate steel tension members.

Modern designers avoid fracture critical details when possible. However, under certain geometric conditions, such as having to space columns very widely to protect an environmentally sensitive site, known fracture critical details such as transverse steel caps are used because they are the only workable design solution currently available.

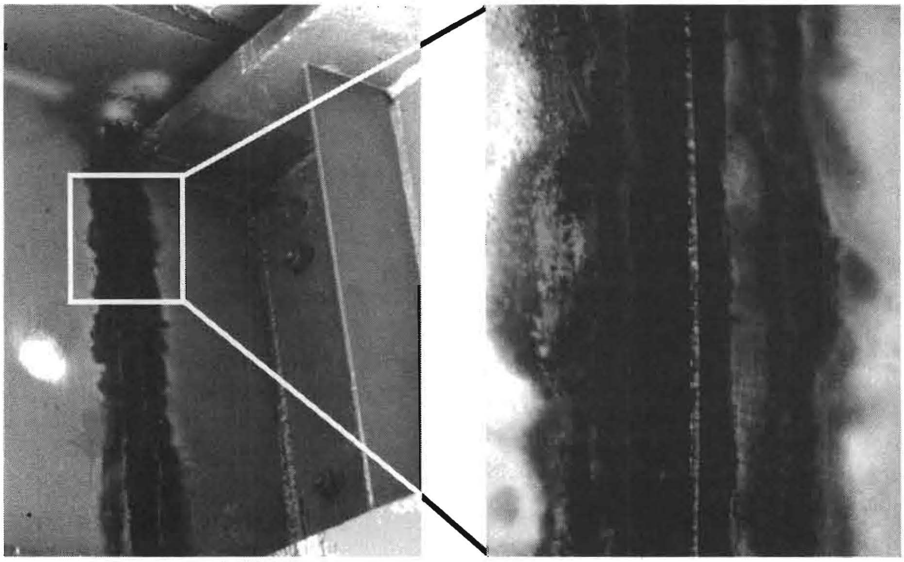

FIGURE 8: Use of dye penetrate to confirm a crack in a girder. (Not fracture critical.)

may reduce the bridge to one-way traffic. The district is responsible for providing a traffic control plan for the fracture critical inspection, as well as the personnel and equipment to implement the plan.

If the bridge has steel box beams 6 to 7 feet in depth and 3 feet wide with the welds on the inside, the box beam must be inspected from the inside. If no inspection port was included in the original design, one must be cut. The inspection port should be opened a day or so in advance of the inspection, to ensure that the air has had a chance to circulate so that it will be fit for the inspector to breathe. Inspection ports should be included as a standard design feature for bridges being planned with steel box beams, since retrofitting is more expensive and less convenient than having them built in in the original construction.

BRINSAP special inspectors rely primarily on visual inspection for cracks. They use dye penetration (Fig. 8) testing to confirm a crack or to check out highly suspect areas such as ends of partial length welded cover plates on girder or beam flanges (Category E or E′) for fine cracking that may not be visible otherwise. If the nature, age, stress range or condition of the structure warrants it, the BRINSAP inspector may decide that other nondestructive tests are necessary. These tests may include ultrasonic testing, magnetic particle, and x-ray screening. The Materials and Tests Division (D-9) performs these tests, and it is BRINSAP's responsibility to schedule them with D-9.

The inspector evaluates all the critical areas and critical details within the areas, documenting the location (Fig. 9), dimension and orientation of cracks as well as the location of suspected cracks and recording the location of excessive corrosion, impact damage and out-of-plane deformation. Traffic conditions are also evaluated.

A representative live load analysis is performed after the inspection to estimate the maximum stress range for the structure. BRINSAP uses the single truck method. This method involves positioning an HS 20 truck in a typical crossing pattern to produce the highest stresses in the critical component and calculating the equivalent stress range following the provisions of AASHTO bridge design specifications. The single truck method provides a valid estimate of stress range because field observation has shown that most large stress cycles are caused by single heavy vehicle passages, rather than by fully loaded vehicles in multiple lanes [Ref. 6, p.130]. An estimated expected fatigue life (remaining life) is then calculated using total number of stress cycles to date (calculated from average daily truck traffic), stress range and the degree of accumulated fatigue damage to date, subtracted from an estimation of total fatigue life.

After the Inspection

Based on the estimated remaining life, the BRINSAP inspectors will determine the frequency of future inspection, from 1 to 5 years. This inspection frequency should be updated by district personnel in the district's BRINSAP record of the bridge under Item 92.1, "Critical Feature Inspection, Fracture Critical Details" [Ref. 1, p.61-62]. Enter a 'Y' in the first space and the frequency in months in the next two spaces.

The BRINSAP inspectors will also make recommendations for repairing, retrofitting or strengthening the bridge. If significant fatigue cracks are found which are perpendicular to the direction of stress, the bridge must be closed until repairs can be made. This is the worst case, and, so far, no such cracks have be discovered during a fracture critical inspection. BRINSAP inspectors may recommend a temporary limitation of the live load if fatigue damage is serious, but the bridge has to be kept in service for a short time before strengthening repairs are made. Usually, however, the

FIGURE 9: BRINSAP inspector documenting a crack.

recommendations are for various types of repairs without any need to limit the live load of the bridge. Correction of frozen movable bearings, rewelding impact damage, and removal of pack rust which is separating plates are typical recommendations. These types of repairs can be contracted out if the district wishes.

SUMMARY

Fatigue cracking resulting in fracture is one of the two main causes of catastrophic bridge failure. The other is scour. District personnel need to be able to identify fracture critical bridges and prioritize them for special inspection by the Bridge Division's BRINSAP Section.

Some broad categories of fracture critical bridges are two-girder steel systems, welded tie arch with box-shaped tie girder, suspension bridges, transverse steel cap bridges, and certain kinds of truss bridges. In general, fracture critical bridges are given priority based on age, stress range, nature of fracture critical detail, and span type, internal redundancy, and damage present.

The BRINSAP Section carries out the actual special inspection with support such as traffic control provided by the district. After the special inspection, the BRINSAP inspectors make recommendations concerning the necessary interval of inspection for that bridge and concerning what repairs need to be made. The district updates the bridge's BRINSAP File record and makes the repairs. The FHWA has mandated that all fracture critical bridges be inspected by March 1992 or states will face possible sanctions.

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STEEL BRIDGES: A SIMPLE, RELIABLE WAY TO MEASURE ACTUAL FATIGUE DAMAGE

What can district personnel do when a condition survey reveals that a steel bridge superstructure scheduled for widening or major rehabilitation does not satisfy existing AASHTO fatigue design requirements? In the past, one of the only options was to replace the structure, due to its low calculated fatigue life remaining, even if the superstructure showed no sign of distress [Ref. 1]. The other main option was to design retrofits that would solve the stress problems — a difficult and often expensive task. With no good way of measuring actual stress history of an in-service bridge, these two options were the only safe way to proceed.

Now, in similar circumstances, the Bridge Division (D-5) has a portable bridge testing system (nicknamed "The Black Box") that can be installed on a steel girder bridge and be used to determine an actual stress history of the bridge under normal traffic loading. A more accurate fatigue life prediction can be made using these field measurements of live load stresses. Since the measured live load stresses are normally much less than the calculated design stresses, most bridges can be shown to have sufficient remaining fatigue life to avoid the need for costly retrofits or unnecessary replacements. The Federal Highways Administration (FHWA) accepts the revised fatigue life estimates that this equipment and these analysis techniques yield.

This bridge testing system was developed through the Texas State Department of Highways and Public Transportation's Research Study 464, ***Estimating Residual Fatigue Life of Bridges***, performed by G. J. Post, K. H. Frank and B. Tahmassebi at the Center for Transportation Research, University of Texas at Austin. The system is portable, self-contained, user friendly and powered by ordinary rechargeable batteries. It can be installed in about six hours by two people using a high-speed grinder from a bucket truck. It can remain unattended on the bridge, automatically recording data from a maximum of eight strain gauge locations, twenty-four hours a day for two weeks. The system is presently limited to steel bridges because no good method exists for testing concrete beams with strain gauges.

The main components of the system are a Campbell Scientific eight channel datalogger and a Data General (DG) portable computer. The datalogger is housed in a specially built aluminum box that can be set on the bottom flange of a girder or on a bent cap. The DG computer provides a simple means of programming the Campbell datalogger to record strains measured using conventional strain gauges or special clamp-on strain transducers. All field wiring is done with prefabricated, silicone-covered modular wiring, so no soldering is necessary. The strain gauges must be glued to bare steel (hence the need for the high-speed grinder); however, the special transducers can be attached to the lower flanges of a girder with ordinary "C" clamps.

The process is menu driven using software developed in the research project, which means that the programs supplied on the DG are specifically written for bridge testing. The system is very flexible with respect to the types of data that can be collected. The Campbell can be programmed to record data continuously while a truck of known weight crosses a bridge, and this data can be used to check analysis results; or the Campbell can be programmed to record and count stress cycles to determine the effective stress range on the bridge for fatigue analysis. Other special tests may also be set up. The research project also developed programs to analyze the data.

The equipment has shown excellent reliability in the field under conditions of rain and of extreme heat. The system initially was used on a number of bridges such as a twin girder bridge in Dallas on east IH-345, an IH-30 bridge in Fairpark and a couple of bridges on US 90 in San Antonio [Ref.2]. The IH-345 bridge had intersecting longitudinal and transverse stiffeners [Ref. 3] that had worse fatigue characteristics than an AASHTO Category E′ fatigue detail. The IH-345 analysis, as well as the IH-30 and US 90 bridges' analyses, found that the fatigue details in question had sufficient residual fatigue life to allow widening without retrofitting the details. Other structures D-5 has

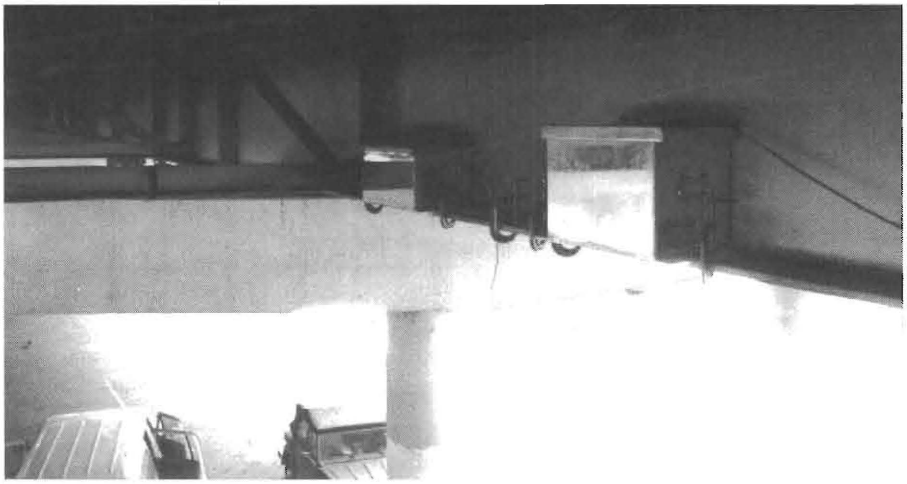

FIGURE 1: The Black Box in place on a structure.

tested to date are: Sulphur Creek Bridge, US 183, Lampasas (See "Aesthetic Bridge Rail Wins Award" below for another research development used on this bridge); Medina River Bridge, US 281, Bexar County; Trinity River Bridge, US 59, Polk County; and Woodrow Wilson Bridge, IH-495, Washington, D. C. (a special project done at the request of the FHWA). These tests also produced favorable results (remaining fatigue lives well over the AASHTO limit of 75 years) and allowed the structures to remain in place without retrofits. In all these cases, the stress ranges were found to be lower than the calculated estimates, but the number of stress cycles were a lot higher. Currently, the bridge testing system is being used to analyze a bridge in Midland which is showing signs of cracking due to out-of-plane bending.

Candidate bridges for the testing system are steel bridges that are scheduled for widening or rehabilitation, have high AADT (particularly high daily truck traffic), and are fracture critical or do not otherwise satisfy existing AASHTO fatigue design requirements. Analysis by this simple and reliable method to determine the actual fatigue damage occurring in a steel bridge due to service stresses can eliminate unnecessary replacement and retrofitting of these structures. The money that would have been sunk into these unnecessary actions can then be applied by a district to other priority bridge problems. For more information, call Gregg Freeby, D-5, (512) 371-5027, Tex-An 254-5027.

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AESTHETIC BRIDGE RAIL WINS AWARD

Sulphur Creek Bridge on US 183 in Lampasas is one of the first bridges to be built using the Texas Type C411 bridge rail developed under Research Study 1185, ***Aesthetically Pleasing Bridge Rails*** (TQ5–3:9). The bridge crosses W. M. Brook Park in a historic part of Lampasas. The C411 rail is designed to be a compatible, architecturally attractive feature in historically significant areas. The city was so pleased with the appearance of the completed rail that the Lampasas County Historical Commission presented a framed bronze star to Luis Ybañez, Bridge Engineer, who accepted it in behalf of the Bridge Division (D-5) of the Texas State De–partment of Highways and Public Transportation (SDHPT). The plaque was given in recognition of the

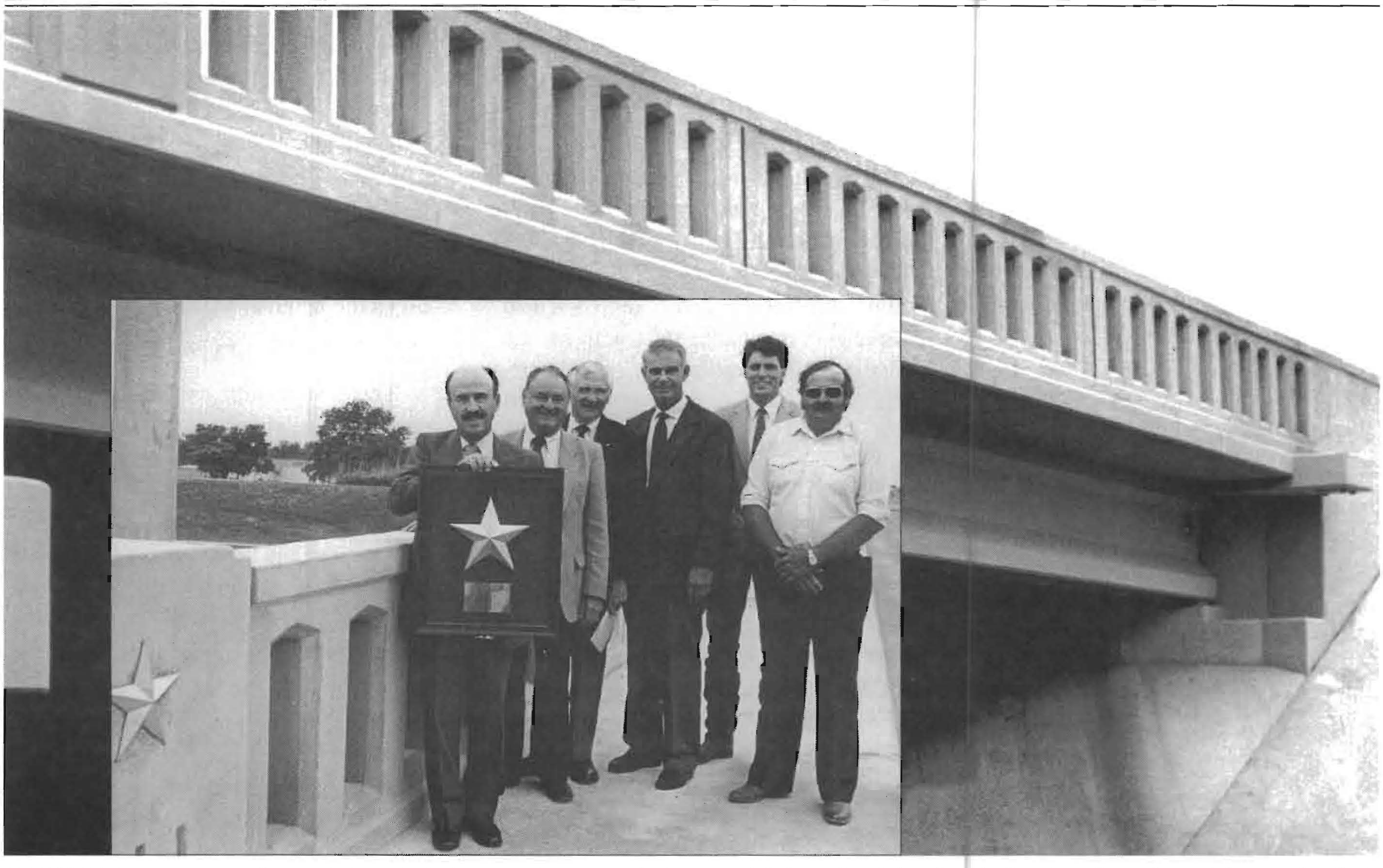

FIGURE 1: Lampasas County Historical Commission Award given at Sulphur Creek Bridge dedication. Inset from left to right: Luis Ybañez, Bridge Engineer, SDHPT; Frank Mayers, FHWA; John Clardy, Chairman, Lampasas County Historical Commission; Lampasas County Judge Norris Monroe; Thomas Bohuslav, Lampasas Resident Engineer, SDHPT; and Jim Skillett, Foreman, Ellis-McGinnis Construction Co. (Photos by Kevin Stillman, Travel and Information Division.)

SDHPT's efforts to support Lampasas County and its heritage. The ceremony took place on Friday, April 26, 1991.

The SDHPT has also been praised by the American Concrete Institute (ACI) for "designing and applying a new railing that meets safety criteria, yet preserves the beauty of older bridges." ACI's Concrete Esthetics Committee adopted the following resolution in December 1990:

The Board of Direction of the American Concrete Institute has noted with appreciation the efforts of the Texas State Department of Highways and Public Transportation to design attractive concrete bridge railings. The department has received favorable mention of this activity in national periodicals such as the ***Wall Street Journal****. Recognizing this effort, the American Concrete Institute Board of Direction commends the state of Texas, its highway department, the officials responsible for attractive concrete designs, and Texas citizens for encouraging highway beautification.*

ACI rightly commends a broad range of people because it was a major cooperative effort among local, state and federal officials (and private citizens) which brought the C411 rail from a design concept in the Bridge Division to existing railings in the Brownwood District. (Construction of the first bridge with the C411 railing was in the city of Brownwood. Lampasas, also in the Brownwood District, has two more C411 construction projects besides the one just built.) Dean Van Landuyt (D-5), who had been the technical coordinator of Research Study 1185 that developed the C411 railing at Texas Transportation Institute, sent preliminary designs out to the district. Lampasas Resident Engineer Thomas Bohuslav and Roadway Designer Tom Judson interested local officials in the aesthetic design. Local officials wanted the superior appearance but could not afford the extra expense (more than twice as much as the standard concrete safety shape). Fortunately, the Federal Highway Administration declared the railing to be experimental and provided 75 percent of the funding for the US 183 bridge renovation. The Ellis-McGinnis Construction Company then built the railing. This kind of cooperative spirit enhances both the community and the department.

The information contained herein is experimental in nature and is published for the development of new ideas and technology only. Any discrepancies with official views or policies of the Texas SDHPT should be discussed with the appropriate Austin Division prior to implementation of the procedures.

SILICEOUS AND LIGHTWEIGHT COARSE AGGREGATE: 24-YEAR PERFORMANCE REVIEW

*(Excerpted by Mohanan Achen from CTR Report 472-3, **A Twenty-Four Year Performance Review of Concrete Pavement Sections Made With Siliceous and Lightweight Coarse Aggregates,** by Mooncheol Won, Kenneth Hankins and B. Frank McCullough)*

INTRODUCTION

Our aging highway system will require efficient maintenance and rehabilitation techniques. Portions of the interstate system are nearing their design life. This would present two choices to a highway designer: build a new highway or rehabilitate the present ones. Considering the financial crisis that the country is facing, the cheaper option, rehabilitation, would be welcome. To understand which rehabilitation or maintenance methods are most effective, a long-term pavement monitoring program is essential. In this article, we will be discussing the results of one such effort on Loop 610 in Houston, Texas.

DESCRIPTION OF THE STUDY

Two short continuously reinforced concrete pavements (CRCP) sections were constructed on the frontage roads of Loop 610 (South) in 1963-1964. The north frontage CRCP section consisted of siliceous gravel coarse aggregate, while the south frontage section consisted of lightweight coarse aggregate. The two-lane concrete test slabs were 6 inches thick on a subbase of 6 inches of cement-stabilized oyster shell. Condition surveys were conducted in 1964, 1974 and 1984. The results of these surveys were combined with data collected in 1988 to develop the performance history of the CRCP pavements.

OBJECTIVES

Three objectives were identified for this project: firstly, evaluate the effect of preformed crack spacing, percent steel, and coarse aggregate type on the long-term performance of the CRCP was to be evaluated; secondly, compare the present conditions of standard aggregate CRCP and lightweight aggregate CRCP to those predicted for one year after construction; and thirdly, compare actual (observed) performance data with mechanistic model predictions made using the CRCP computer program. The ultimate goal is to increase understanding of pavement performance and pavement modelling systems.

DESIGN OF EXPERIMENT

With regards to the ratio of cross-sectional area of the steel to the concrete area, the two test areas in the project had 0.3 percent, 0.4 percent, and 0.5 percent longitudinal reinforcing steel in the standard CRCP and 0.3 percent and 0.4 percent steel in the lightweight CRCP. The transverse steel for both lightweight and standard consist of 1/2-inch bars at 32-inch centers. An effort was made to maintain a constant bond area to volume of concrete ratios for all sections.

The standard CRCP design used Item 366 of the Standard Specifications. The specifications for the lightweight aggregate concrete called for a cement ratio of 5-1/2 sacks per cu. yd., a six to nine percent air content by volume, a slump of 2.3 inches, ASTM C330 lightweight aggregate, a maximum unit weight of 55 lb per cubic feet and natural sand as fine aggregate. The maximum size of the synthetic lightweight aggregate was approximately 3/4 inches while, the maximum size of siliceous river gravel used as conventional aggregate was approximately 1-1/2 inches.

The design for standard and lightweight CRCP was supposed to provide optimum crack spacing. Preformed cracks were required to provide different crack spacings. To limit end movements, the pavement ends were anchored by two transverse lugs. All test sections in each slab were placed in one working day to minimize the effects of weather conditions. The original design for the pavement sections had anticipated 1.5 million 18-KSAL but at this present time, the actual loadings have exceeded the design loadings by four times the original value.

MEASUREMENTS

Three structural variables, namely crack width, crack spacing, and steel stress, influence each other in the design of CRCP. Only crack spacing can be measured easily and accurately. Crack spacing measurements were conducted in 1964, 1974, 1984 and 1988, whereas crack width and steel stress were only measured in 1964 and 1988 respectively. Spalling and punchouts, both minor and severe, were measured in 1988. Deflections were measured using the Basin Beam in 1964 and using the Dynaflect in 1984. ACP and PCC patches were surveyed in May 1988.

24-YEAR PERFORMANCE ANALYSIS

This study provided the unique opportunity to evaluate the performance of a CRCP pavement throughout an entire life cycle. Pavement performance was evaluated in terms of crack spacing, deflection and surface conditions.

Deflections

The 1974 and the 1984 sets of deflection data display some different trends.

1974 — Deflections vary inversely with percentage of longitudinal steel for all conventional aggregate sections except for the 0.5 percent steel, 5-foot preformed crack spacing. Lightweight aggregate pavement sections with low modulus of elasticity concrete were discovered to deflect more than conventional concrete sections, except for deflection at cracks for 0.4 percent steel sections. The optimal preformed crack spacing for conventional concrete pavements was determined to be 5 feet, based on the fact that deflections at cracks for 5-foot sections were less than those for 8-foot sections. For lightweight aggregate sections, an 8-foot preformed crack spacing seems to be optimal, despite the fact that 20-foot preformed crack spacings generally performed better than the 8-foot preformed crack spacings. This is because 20-foot and 8-foot preformed crack spacing sections tend to have an 8-foot mean crack spacing.

1984 — These data have reverse deflection trends with relation to percentage of longitudinal steel. Deflections seem to increase with higher percentages of steel. The exceptions were deflections at cracks on lightweight aggregate sections with 8-foot preformed crack spacing and deflections at midspan for lightweight aggregate sections with 20-foot preformed crack spacing. The other trends were similar to 1974.

In general, the heavily reinforced sections are stiffer than the lightly reinforced sections for a certain time period, after which the data is random. Lightweight aggregate sections deflect less than conventional aggregate sections, but the lightweight aggregate sections seemed to lose some of their stiffness between 1974 to 1984. Conventional aggregate sections seem to have a design life between 10 and 20 years of age.

Transverse Cracking

The transverse crack analysis of both conventional and siliceous river gravel aggregate sections revealed the inability of 0.3 percent steel to withstand shearing forces. Sections with 0.4 and 0.5 percent steel have average crack spacings of 2 to 3 feet after 24 years of service. Both 0.3 and 0.4 percent lightweight aggregate steel sections have an average crack spacing of around 8 feet. All preformed locations developed transverse cracks after a period of one year or after about 500,000 18-KSALs.

Transverse cracking also developed between preformed locations. The average transverse crack spacing tends to level off at about 8 feet for the lightweight-aggregate and 2 to 3 feet for the conventional aggregate sections. Applying the preformed crack technique seems to eliminate pronged or Y cracks. In lightweight sections, there seems to be uniformity in the spacing of the straight preformed cracks at the 20-foot spacing. Larger preformed spacings tend to provide larger average crack spacings for a longer time.

Condition Surveys

Spalling. The conventional aggregate or siliceous river gravel sections had more spalling as compared to the lightweight aggregate sections where no spalling was noted. The conventional 0.3 percent steel sections had more spalling than the other sections. The amount of spalling seems to increase with a decrease in the percentage of steel. Spalls occurred at both the preformed cracks and at cracks between the preformed spaces.

Punchout. A severe punchout is defined as a relatively large punchout accompanied by spalling and/or movement of the formed block. No severe punchouts were found in the lightweight sections. Severe punchouts occurred in conventional aggregate sections containing 0.3 percent steel with 8-foot preformed crack spacing and those containing 0.4 percent steel with 5-foot preformed crack spacing.

Patched Areas. A patched area is defined as square feet of patched area per 100 feet along the 24-foot wide pavement. Patched areas are basically punchouts which have been repaired. Generally, the area patched increased as the steel percentage decreased. Lightweight aggregate sections did not require patches. Only in 0.3 percent steel sections did preformed crack spacings have any effect on the patched area. Eight-foot preformed spacing sections had less patched area than 5-foot preformed spacing sections.

COMPARISON WITH RESULTS FROM MECHANISTIC MODELS

Mechanistic analyses for this study were accomplished with the help of the CRCP program. This program details structural responses of the CRCP for environmental conditions and wheel loads as a function of time. The input parameters for the computer program CRCP consist of (1) material properties, (2) steel and thickness design, (3) environmental conditions, and (4) traffic loading conditions. The program predicts crack width, concrete and steel stresses, and mean crack spacing. In lightweight aggregate sections, there is a large discrepancy between predictions and actual values. The higher values in long-term creep of lightweight concrete are believed to reduce the concrete volume change stress and also contribute to larger mean crack spacings. Since the computer does not consider creep, the discrepancy can be expected. The predictions regarding the conventional aggregate sections containing 0.4 percent and 0.5 percent steel were accurate, while those for 0.3 percent were not.

CONCLUSION

Lightweight aggregate sections had performed superbly during the test period. These sections looked good, had no failures and possessed relatively large crack spacings. On the contrary, the conventional aggregate sections needed some rehabilitation work because of spallings and punchouts.

The preformed crack technique tended to provide a straighter crack and this helps reduce early spalling and punchouts. This technique seem to have a positive effect on N or Y cracking and reduces the longitudinal meandering of the transverse cracks. The preformed crack technique tends to reduce the number of very small crack spacings by delaying intermediate cracking and focusing it between the preformed locations. Longer preformed spacings tend to provide larger average crack spacings for longer time periods like ten years. This study reinforces the cost effectiveness of the preformed crack technique.

Study 472 proves that preformed crack spacing length depends upon the concrete material properties like coarse aggregate type. The conventional aggregate or siliceous river gravel sections produce smaller crack spacings as compared to lightweight aggregate sections.

Performance of river gravel sections improved with higher steel percentages. Three-tenths percentage of steel is not sufficient for siliceous gravel sections, but works for lightweight aggregate sections. The Texas SDHPT is still encouraged to use six-tenths percentage of steel in their CRCP. Mean crack spacings did not seem to be affected by variances in either the amount of reinforcing steel or the preformed crack spacings; ratio of bond area to concrete volume is a better determinant of mean crack spacing.

GEOMEMBRANE USE IN TRANSPORTATION SYSTEMS

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According to ASTM, a geomembrane is defined as: "An essentially impermeable membrane used with foundation, soil, rock, earth or any other geotechnical engineering-related material as an integral part of a human-made project, structure, or system." Most geomembranes are thin sheets of flexible polymeric materials manufactured by one of the following three methods:

- Extrusion — nonreinforced.
- Calendering — nonreinforced or reinforced.
- Spread coating — reinforced.

The reinforced geomembranes have a fabric scrim or fabric substrate integrated within the separate piles or beneath the surface coating. Subsequent factory fabrication of geomembrane sheets leads to panels that are made as large as possible to expedite field placement while minimizing field seaming.

Concerning polymer types, all geomembranes are made from blended compounds of primary resin(s) and other ingredients. For example, Haxo [Ref. 1] polymeric liners could be thermoset, thermoplastic, or semicrystalline and could be composed of polymer or alloy, with varying percentages of oil or plasticizer, fillers (carbon black, inorganics), antidegradants, and crosslinking agents (inorganic, sulfur). Table 1 in dictates the major generic types of geomembranes currently used in North America.

Geomembrane use in subsurface construction work has grown rapidly, [Ref. 2] with current annual North American sales about 33 million sq yd. Major application areas include transportation, 10 percent of sales; environmental, 80 percent (liquid containment 22 percent, solid containment 53 percent, and vapor containment 5 percent); and geotechnical, 10 percent. The major types of linear materials include PVC, 24 percent; CPE/CSPE, 25 percent; HDPE, 41 percent; and miscellaneous, 10 percent. Thermoset materials are seldom used today.

PVC is most used for transportation–related applications, followed slightly by CPE/CSPE, and finally HPDE. The reason for this comparative lack of HDPE use is that chemical resistance is not usually a compelling criterion (water versus leachate) and ease of construction takes precedence.

GEOMEMBRANE PROPERTIES AND DESIGN METHODS

As with any engineering material, a geomembrane's properties must be measured in an organized and quantifiable manner. Fortunately, ASTM has taken a leadership role in this regard by forming Committee D-35 on Geosynthetics. Carroll [Ref. 3] gives a historical perspective of ASTM's involvement as well as descriptions of other important standards groups, e.g., AASHTO, Task Force # 25, and others. Individual states' involvement in geomembranes is limited; few mention geomembranes in their regularly published specifications.

The major properties of geomembranes can be broken down by category, e.g., physical, mechanical,

TABLE 1: Categories and types of geomembranes currently used.

Category	Acronym	Name
Thermoset	IIR	Butyl rubber
	EPDM	Ethylene propylene diene monomer
Thermoplastic	CPE	Chlorinated polyethylene
	CPE-A	Chlorinated polyethylene alloy
	CSPE	Chlorosulfonated polyethylene
	EIA	Ethylene interpolymer alloy (XR-5)
	PVC	Polyvinyl chloride
	PVC-OR	Oil resistant polyvinyl chloride
Semi - crystalline	HDPE	High density polyethylene
	HDPE-A	High density polyethylene alloy
	MDPE	Medium density polyethylene
	VLDPE	Very low density polyethylene
	LLDPE	Linear low density polyethylene

and so forth, in a way that a total perspective of a specific geomembrane can be obtained. But generalities about typical properties are difficult to make. Table 2 is a recent compilation [Ref. 2] that illustrates the wide ranges of properties and values available. A particular value's importance within this range will become apparent during design.

Designing with geomembranes should focus on its primary function and the related mechanism. As such, a traditional factor of safety equation can be formulated:

$$FS = \frac{\text{Allowable (test) Property}}{\text{Required (design) Property}}$$

A test method that adequately models a real situation gives the allowable property in the above equation directly, e.g., thickness, tensile strength, puncture resistance, etc. If the test methods are not accurate, a reduced value becomes necessary. This can sometimes be obtained by a semi-empirical technique, as in Koerner [Ref. 4].

The required property in the above equation is generally obtained by a design model, mostly adapted from geotechnical engineering analysis. For geomembranes in environmental linear and cover situations, a design guide by Richardson and Koerner [Ref. 5] is available. Unfortunately, there is no such design guide for transportation applications, per se, although the literature is growing. A lower limit for the required properties in the equation should focus on installation survivability demands placed upon the candidate geomembrane. Table 3 provides information on various properties as a function of anticipated demands placed on the geomembrane. We must emphasize, however, that these minimum values cannot replace rational design-generated values. If such design values are higher than those listed in Table 3, they must take precedence.

TABLE 2: Major properties of geomembranes together with typical values [Ref.2].

	Approximate Range of Values	
Category and Property	**Standard Units**	**International Units**
Physical		
Thickness	10—100 mils	0.25—0.25 mm
Specific Gravity	0.9—1.5	0.9—1.5
Weight (mass per unit area)	20—100 oz/yd^2	600—3000 g/m^2
Water Vapor Transmission	2—20 x 10^{-14} lb/ft^2-24 hr	1—10 g/m^2-24 hr
Mechanical		
Tensile Strength at Yield		
Unreinforced	5—25 lb/in.	1—5 kg/cm
Reinforced	25—100 lb/in.	5—20 kg/cm
Tensile Strength at Break		
Unreinforced	5—25 lb/in.	1—5 kg/cm
Reinforced	10—30 lb/in.	2—6 kg/cm
Elongation at Yield		
Unreinforced	20—100%	20—100%
Reinforced	10—30%	10—30%
Elongation at Break		
Unreinforced	100—500%	100—500%
Reinforced	70—250%	70—250%
Modulus of Elasticity		
Unreinforced	500—3,000 lb/in.	3.5—20 MPa
Reinforced	5,000—20,000	35—140 MPa
Tear Resistance		
Unreinforced	4—30 lb	2—15 kg
Reinforced	20—100 lb	10—50 kg
Impact Resistance		
Unreinforced	0.5—15 ft-lb	0.05—2 kg-m
Reinforced	17—50 ft-lb	2—7 kg-m
Puncture Resistance		
Unreinforced	10—100 lb	5—50 kg
Reinforced	50—500 lb	25—250 kg
Soil to Linear Friction- (% of soil friction)	50—100%	50—100%
Seam Strength (% of linear strength)	50—100%	50—100%
Chemical		
Ozone resistance	Varies with liner and location	
Ultraviolet light resistance	Varies with liner and location	
Chemical resistance	Must be specifically evaluated	
Thermal		
Hot climates or conditions	Usually no problem regarding material	
Cold climates or conditions	Decreases ductility, difficult to seam	
Biological		
Stability to microbe attack	Usually no problem	
Durability		
Water absorption	0—30%	0—30%
Aging	No standard procedure to evaluate over long time periods	

SPECIFIC APPLICATION AREAS

Geomembranes have been used in numerous transportation-related applications. While specific uses often do not cover extremely large areas, they do solve meaningful and oftentimes difficult problems. Several uses are described below (including appropriate references).

Preventing Upward Groundwater Movement in Railroad Cut

Described by Lacey [Ref. 6], this project used a scrim reinforced CSPE geo-membrane on the soil subgrade and beneath the railroad ballast. A needle-punched nonwoven geotextile was used above the geomembrane to resist puncture from the ballast. Waterproof seals were required at each concrete cantilever retaining wall paralleling the cut. These particular details are critical to the system's total performance. It should be cautioned though, that high pore water pressures often occur in railroad applications and pressure relief wells may be required.

Waterproofing Transportation Tunnels

Water seeping into transportation tunnels is a constant problem. When tunneling in rock and the excavation is made by blasting, a shotcrete layer is often placed as soon as possible. This has been called the "New Austrian Tunneling Method." By attaching a thick needle-punched nonwoven

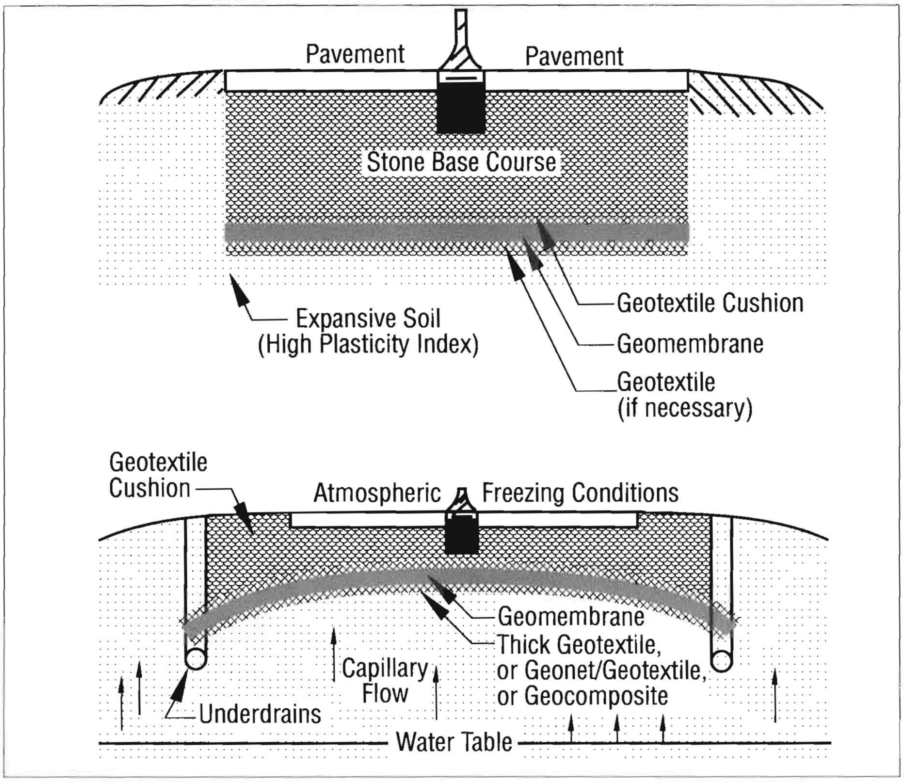

FIGURE 1: Geomembranes used to control expansive soil and prevent frost heave.

geotextile to the shotcrete, followed by a geomembrane and the final concrete liner, an excellent waterproofing system is achieved. The geotextile intercepts seeping water and directs it into appropriate underdrains. Frobel [Ref. 7] has used PVC geomembranes for this type of application.

Preventing Contamination in Railroad Refueling Areas

A spreadcoated butyl geomembrane on a needle-punched nonwoven geotextile has been used to prevent subsurface diesel fuel contamination. The concept was first presented by True Temper, Inc. The sides and bottom of the cross section were covered in this manner, while the surface has only a geotextile covering. The enclosure requires outlet drains to remove collected diesel fuel. Note that the geotextile on the geomembrane faces inward against the ballast to provide necessary puncture protection.

Moistureproofing Railroad Subgrades

Soil subgrade pumping by heavy, cyclic loads is a common railroad problem that rapidly contaminates ballast. Ayres [Ref. 8] describes geomembrane use to prevent this problem. A geotextile cushion above the geomembrane for puncture resistance is again used. As noted before, high pore water pressures created in many railroad environments may require pressure relief wells beneath the geomembrane.

Control of Expansive Soils (Vertical Infiltration)

Expansive soils are found in many areas. When these soils absorb water, they can swell and increase their volume substantially. Eliminating downward moving moisture with a geomembrane has been successfully used. A geotextile cushion is used above and, depending on the quality of the subgrade, sometimes below the geomembrane. Sheffield and Steinberg [Ref. 9] discuss applications of this method.

Prevention of Frost Heave

Upward migration of groundwater within a capillary zone may meet an elevation in the soil profile where freezing conditions exist. When this occurs, ice lenses can grow continuously, lifting everything above them. A possible remedial scheme uses a geomembrane barrier with a geotextile or geonet drain beneath it. If a geonet is used, its underside must

TABLE 3: Recommendated minimum properties for general geomembrane installation survivability [Ref. 2].

	Required Degree of Survivability			
Property and Test Method	**Low**	**Medium**	**High**	**Very High**
Thickness (D-1593) mils (mm)	20 (0.50)	25 (0.63)	30 (0.75)	30 (0.75)
Thickness D-882 (1.0" (25mm) strip) lb/in. (kN/m)	30 (5.2)	40 (7.0)	50 (8.7)	60 (10.5)
Tear (D-1004 Die C) lb (N)	5 (22)	7.5 (33)	10 (45)	15 (67)
Puncture (D-3998 mod.) ft-lb (J)	20 (90)	25 (110)	30 (130)	35 (160)
Impact (D-3998 mod.) ft-lb (J)	10 (7)	12 (9)	15 (11)	20 (15)

Notes:
"Low" refers to careful hand placement on very uniform well-graded subgrade with light loads of a static nature – typical of vapor barriers beneath building floor slabs.
"Medium" refers to hand or machine placement on machine-graded subgrade with medium loads – typical of canal liners.
"High" refers to hand or machine placement on machine-graded subgrade of poor texture with high loads – typical of landfill liners and covers.
"Very High" refers to hand or machine placement on machine-graded subgrade of very poor texture with high loads – typical of reservoir covers and liners for heap leach pads.

have a lightweight geotextile filter for protection [Ref. 2]. The geotextile or geonet drain must connect to an underdrain beyond the limits of the concerned area. The underdrained could be a synthetic edge drain composite.

Preventing Enlargement of Karst Sinkholes – Another Theoretical Application

Many limestone formations are reactive when water contacts them. This well-known solution phenomenon is called "karst" topography or "sinkhole" formations. Preventing further enlargement of existing sinkholes might be accomplished with a geomembrane (with geotextile protection) that keeps rainwater and snowmelt from entering the soil subgrade.

Protecting Frost Sensitive Soils

The concept of a membrane encapsulated soil layer (MESL) has been pioneered by the Cold Regions Research Laboratory of the Corps of Engineers [Refs. 10-11]. Placed and maintained at the soils' optimum water content, the encapsulated soils are suitable for light roadways. Without encapsulation, however, the soils would become saturated and lose strength. The moisture barrier needed to prevent this can be among those listed in Table 1, but is usually a nonwoven geotextile impregnated by an asphalt emulsion or elastomer spray. Various techniques are described by Meader [Ref. 12].

Protecting Friable Soils

The same MESL concept has been used to preserve the moisture content of friable soils in arid regions [Ref. 13]. This is the inverse problem from frost sensitive soils; drying causes friable soils to fall apart. With this technique the encapsulated zone depth will probably be deeper than with frost sensitive encapsulated soils.

Control of Expansive Soils (Horizontal Infiltration)

Moisture entering expansive soils beneath pavements also occurs horizontally. Vertical barriers can be deployed as described by Sheffield and Steinberg [Ref. 9] and in the literature. The geotextile/geomembrane "curtains" can be installed with new pavement or with pavement overlays.

Secondary Containment of Underground Storage Tanks

Using a hydrocarbon resistant geomembrane sandwiched between two geotextiles, a secondary liner system with internal leak detection (in the bedding stone) is formed. Such systems have been marketed by at least two organizations, Seamans Inc., Millersburg, Ohio and MPC, Inc., Chicago, Illinois. A different scheme, by Total Containment, Inc., Exton, Pennsylvania uses a geonet leak detector around the tank, with an encapsulated geomembrane on the outside.

Wall Waterproofing Systems

We can envision various schemes to keep surface water from seeping behind retaining walls. This would help prevent possible corrosion or provide relief from hydrostatic pressure. Many types of geomembranes are possible, but all must be adequately protected if they are at or near the surface and if heavy loadings are anticipated.

Other than the applications that require chemical resistance, the uses cited above provide barriers to surface water or groundwater. As such, chemical resistance should not be formidable concern and most polymer types listed in Table 1 should be adequate. Other conditions such as mechanical properties, seamability, and cost probably take precedence.

Many applications require that the geomembrane be protected against punctures. This can be provided by placing a geotextile against the geomembrane, or by a geotextile/geomembrane composite made by spread coating or post-fabrication bonding. Several composites are commercially available. In some cases, a geotextile is required on both sides of the geomembrane.

The generally preferred geotextile in these applications is a relatively thick needle-punched, nonwoven type, where a cushioning action provides puncture resistance. While such a

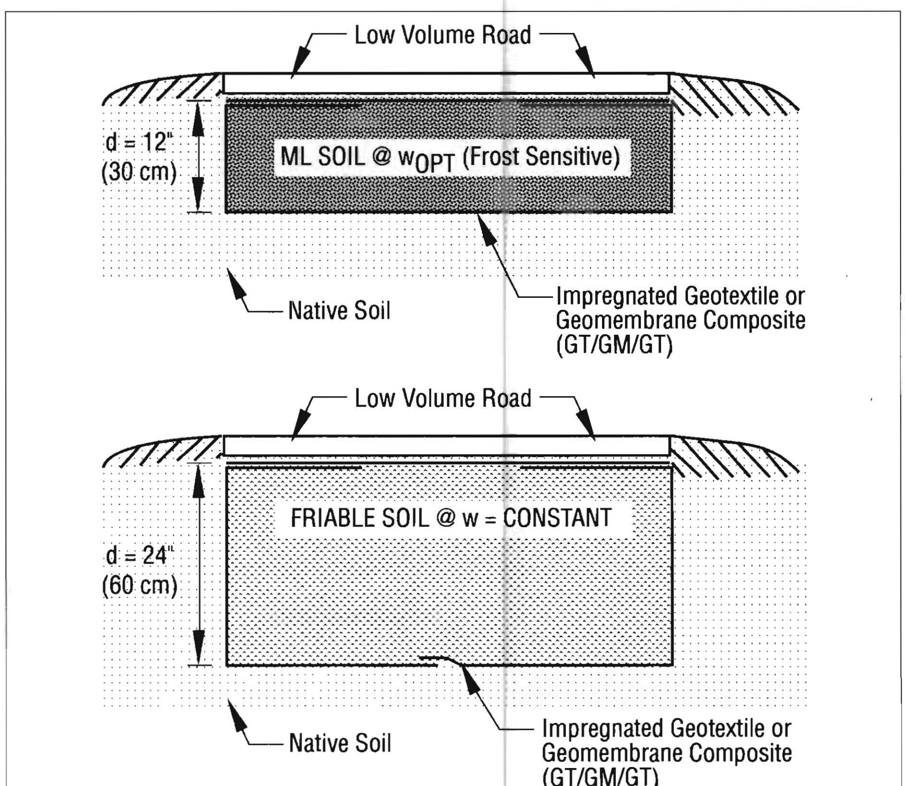

FIGURE 2: Geomembranes installations for maintenance of a desired water content.

mechanism is certainly obvious, other geotextiles might also be feasible, by virtue of their load spreading capability. Further investigations in this regard seem warranted.

Geomembrane seams are always a concern from both strength and moisture tightness considerations. These concerns are site specific and often "absolute" tightness is not necessary. In this regard, seaming bonded geomembrane/geotextile composites is often not a detriment, and they can be mechanically seamed, e.g., by sewing, or sometimes merely overlapped.

In conclusion, the use of geomembranes for subgrade applications is an exciting and growing field. Many opportunities exist for all segments of the profession and related industries. With a broad based educational effort will come widespread familiarization and continued strong growth for geomembranes in the future.

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BEN AHIN BRIDGE

An unusual construction technique is underway in Belgium, where the Ben Ahin Bridge, with a single off-center pylon, is being built to cross the River Meuse. The structure, with a main deck 294 meters long, is being built parallel to the banks of the river, and when the deck is complete the 16,000 ton structure will be rotated through 70 degrees to cross the river.

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