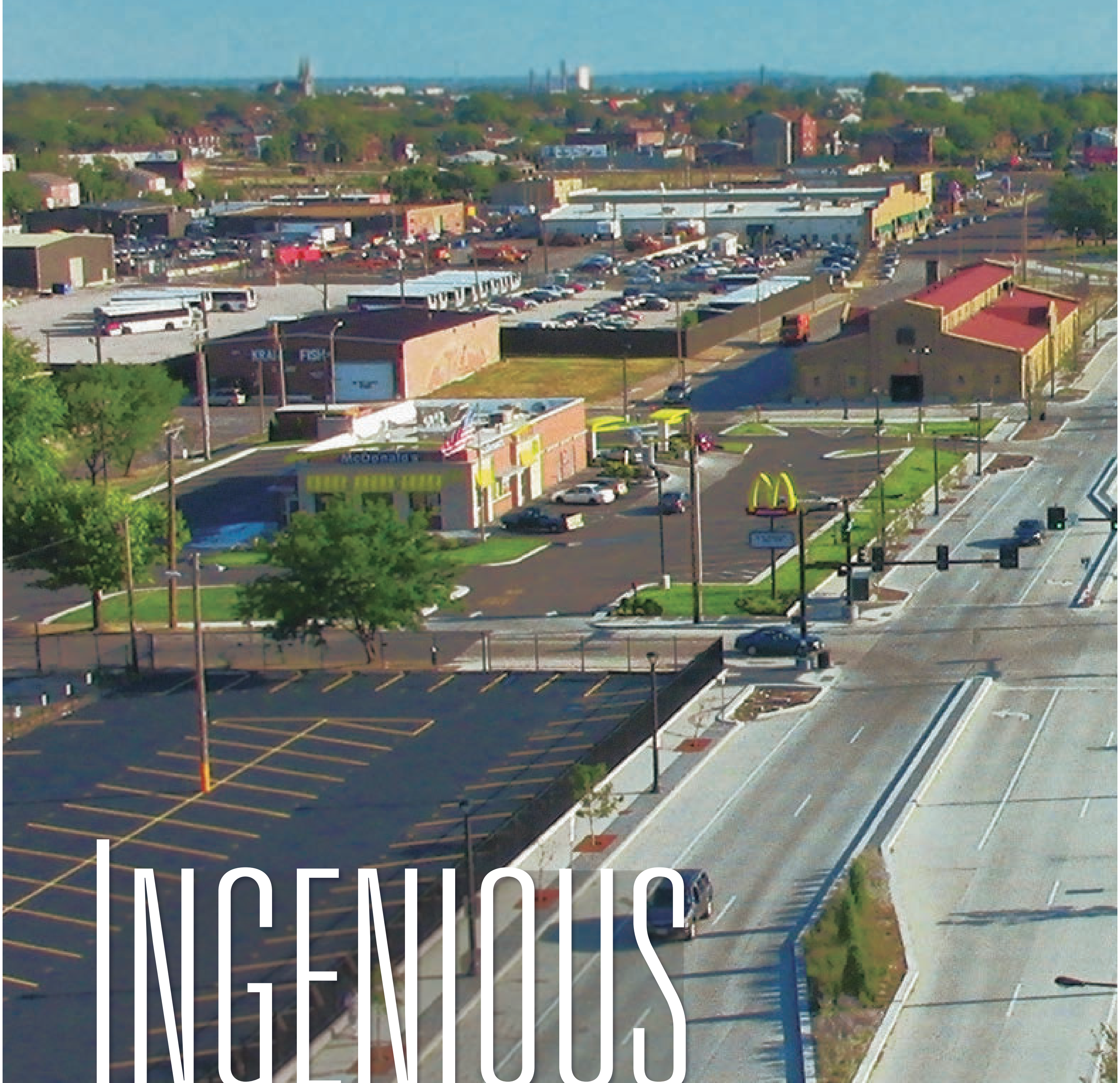

INGENIOUS INFILL

Before a deteriorating bridge from the 1930s could be removed in downtown St. Louis, project designers had to devise a way to fill in the 30 ft deep excavation beneath the ground-level structure without imposing lateral loads on adjacent buildings. The novel solution involved using blocks of lightweight expanded polystyrene near the buildings and traditional soil fill within the rest of the excavation.

By Justin Anderson, P.E., M.ASCE, Patrick Poepsel, P.E., M.ASCE, and Kevin Kriete, P.E., M.ASCE

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The newly refinished, earthquake-resistant section of North Tucker Boulevard, which is spurring such commercial development as a new McDonald's restaurant, eliminates the safety concerns that plagued the former roadway, *inset.* Barriers had been used to block certain problematic portions of the former roadway.

HDR, INC., BOTH

CONSTRUCTED IN DOWNTOWN St. Louis in 1931, the North Tucker Boulevard Bridge facilitated the movement of vehicular and pedestrian traffic along North Tucker Boulevard while a passenger railway operated within a 30 ft deep trench beneath it. However, by the mid-1950s passenger rail service had ended along the route. By the early 1980s nearly all rail activity had ceased at the site, and today there is no rail service at all along the route. During the past 20 years the North Tucker Boulevard Bridge has undergone significant deterioration and has required numerous repairs. Faced with safety concerns and the prospect of additional costly repairs, the City of St. Louis decided to remove the bridge, backfill the excavation to ground level, and support the roadway at grade. Because of the proximity of several venerable buildings, the project designer—HDR Engineering, Inc., of Omaha, Nebraska—recommended filling in the trench with a combination of expanded polystyrene (EPS) and soil fill. Also known as geofoam, the lightweight EPS would be used adjacent to the buildings so as to avoid imposing lateral loads on the structures. A 6 ft thick soil layer would be placed on top of the EPS fill to accommodate buried utilities and support tree growth.

Because the site is situated within the New Madrid Seismic Zone, the response of the EPS and soil to strong ground motions was a significant element of the design. The project design had to address numerous other challenges as well, including analyzing and predicting stresses and strains throughout the free-standing EPS structure during traffic loading and earthquake shaking using finite-element methods. It would also be necessary to design an underdrain system to collect groundwater and prevent uplift on the EPS from the buildup of groundwater, minor water line breaks, utility leaks, or surface water infiltration. Other challenges included the design of conventional mechanically stabilized earth walls and associated ground improvements and the design of elements to reduce downward drag on existing foundations that would receive earthen fill. Meanwhile, the 6 ft of cover soil on top of the EPS would produce a top-heavy structure that would present significant design challenges, particularly with regard to interactions between soil and structure during seismic events and predictions of deformation.

The most significant problem areas of the bridge were adjacent to the expansion joints, along the curb lines, and along the sidewalks on each side of the bridge deck.

The rail system had been built in a large open excavation. North Tucker Boulevard was reconstructed above it on a series of bridge units whose supports were independent of the existing buildings, many of which predated the project. Portions of the roadway sidewalk were found to span from the bridge deck to support corbels on the adjacent buildings. It is important to note the close proximity of the excavation slopes to the existing buildings, as well as the early use of pile-supported bridge caps and precast-concrete retaining walls to separate the bridge loading from the existing structures. The original excavations were inclined at about 1.5:1 (horizontal to vertical) and have remained at these slopes to the present.

Before design work began on the reconstruction project, the North Tucker Boulevard Bridge had been rated in the inspection program that uses the Federal Highway Administration's National Bridge Inspection Standards. The rating scale ranges from 0 to 9, 0 denoting a closed bridge and 9 a structure in excellent condition. The deck and substructure were found to exhibit signs of advanced section loss, deterioration, and spalling, earning a 4. The superstructure was found to exhibit loss of section, deterioration, and spalling that had affected primary structural components and thus was given a 3. The bridge had posted weight limits of 22 tons for single-axle trucks, 39 tons for double-axle trucks, and 44 tons for double-trailer trucks.

The most significant problem areas of the bridge were adjacent to the expansion joints, along the curb lines, and along the sidewalks on each side of the bridge deck. This deterioration resulted mainly from deicing salts and chemicals that were used during winter months and drained through the expansion joints and leached through construction joints in the deck. This corrosive solution then attacked the unprotected steel superstructure, a process exacerbated not only by the dampness of the area beneath the bridge but also by connection details that did not allow proper drainage or easy inspection of the U-shaped connections referred to as expansion pockets that were used to support the beams while allowing thermal movement at the expansion joints.

The substructure units typically comprised cast-in-place concrete foundations supporting steel frame bents consisting of columns and main cross-girders. The steel columns and lateral bracing of the bents were typically found to be in good condition. While minor corrosion on the columns was found, there was no significant section loss. Some of the girders still had an intact coat of paint, although most had visible rust. The main steel cross-girders of the frame bents supporting expansion joints all exhibited differing degrees of corrosion. Most of these girders were heavily rusted on the expansion side, and half of them showed additional minor corrosion on the fixed side. At the worst joints, the girders showed evidence of significant section loss. The steel expansion pockets were filled with dirt and rust, which trapped salt-saturated water. Several of the pockets also exhibited significant section loss.

The city had made numerous repairs to the bridge, primarily in the 1990s. Major repairs largely consisted of removing and replacing severely deteriorated portions of stringers near the expansion joints. In more severe cases, the entire beam and expansion pockets were replaced. In addition to other extensive repairs, the city installed temporary shoring at numerous locations to support the deteriorated stringers and prevent differential displacement of the bridge deck. These temporary supports were a cost-effective approach to keep the bridge functional until funding became available for total replacement. The steep railway excavation slopes

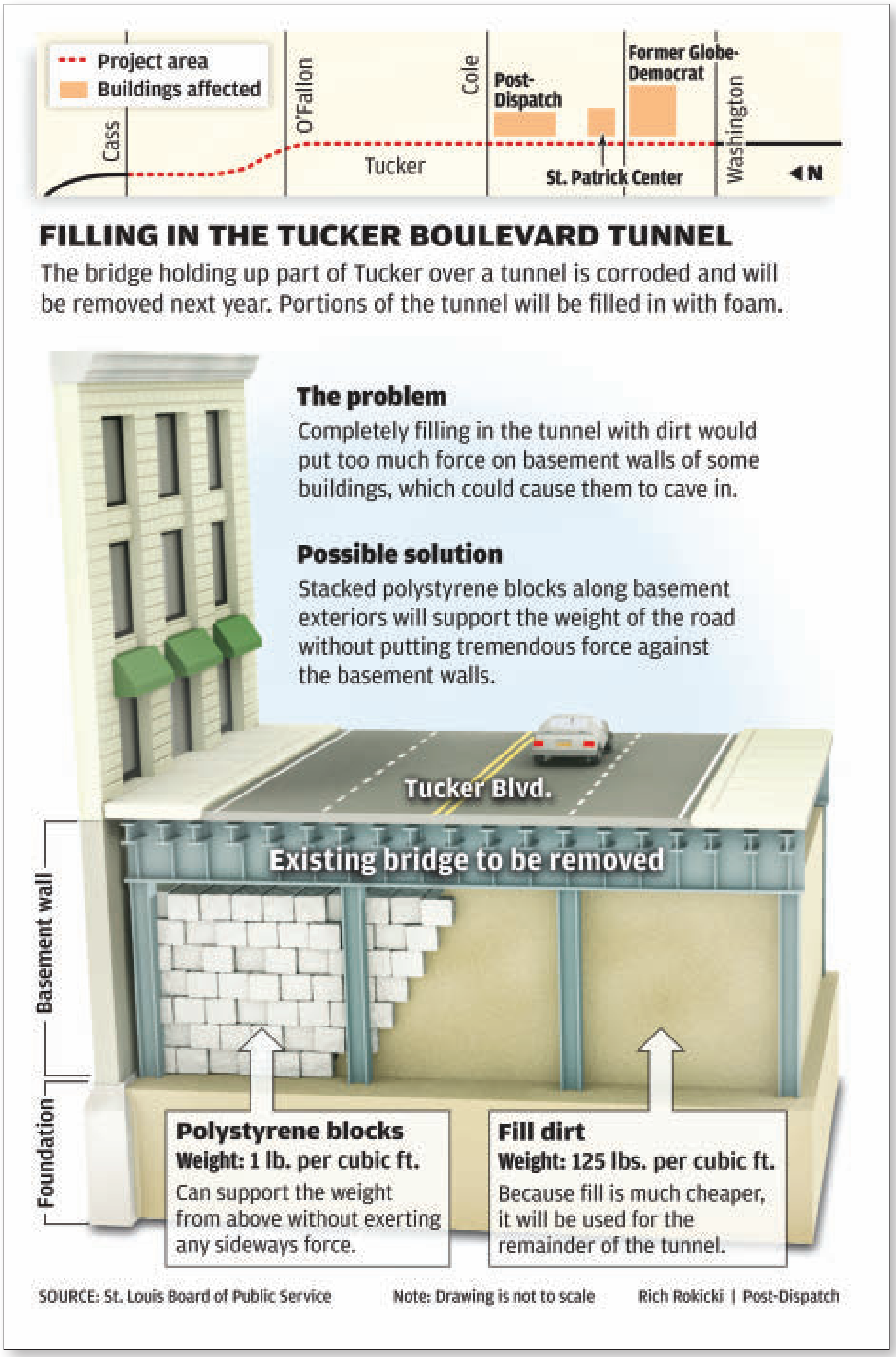
Given the presence of buildings immediately adjacent to the proposed roadway embankment, carrying out future inspections, maintenance, and repairs would be difficult or impossible without removing large portions of the embankment.

performed reasonably well, with localized instances of slope movement or recurring maintenance issues such as sloughing or surface erosion. Because of its continued structural deterioration, the bridge was ultimately closed to traffic before the replacement design was completed.

In late 2007 St. Louis's Board of Public Service hired HDR as the prime consultant to conduct a feasibility study to develop options for removing the North Tucker Boulevard Bridge and filling in the railroad excavation. The scope of the project was to provide design services for the demolition and removal of the North Tucker Boulevard Bridge and to restore the roadway embankment profile without damaging the existing buildings and utilities. To achieve that goal, several challenges and impediments had to be overcome. The primary design criterion was to provide a solution that would offer a 100-year design life. Given the presence of buildings immediately adjacent to the proposed roadway embankment, carrying out future inspections, maintenance, and repairs would be difficult or impossible without removing large portions of the embankment.

Three additional design criteria were specified:

- Restrict the lateral movement of the EPS and soil system;
- Maintain the bearing pressures on the native soil within allowable values (typically less than 2,000 psf);
- Predict the stresses and strains of the EPS so that the response of the material would remain elastic.



The primary site challenges included limited construction access points; a tight, irregularly shaped construction area; an inflexible construction schedule; and the need to reduce the extent to which the roadway and sidewalks were closed, as such closures would limit access to local businesses.

Surveying the site proved difficult, as the large, irregularly shaped project area had numerous structures located both aboveground and belowground that needed to be surveyed. Traditional surveying techniques alone would have consumed much of the already limited project schedule and might not have provided the level of detail necessary to successfully and accurately complete the design. Therefore, lidar surveying was used in combination with a field survey at street level.

Lidar surveying can quickly and accurately measure the shape of the ground surface and can differentiate between natural and man-made features. By merging Global Positioning System and other technologies, lidar mapping can map the topography of a site and the three-dimensional positioning of obstacles, objects, and equipment within the area. For the North Tucker Boulevard Bridge replacement project, these data were used to develop three-dimensional topography, which meant greater accuracy in the design process and in estimating quantities and construction costs.

The generalized subsurface profile along North Tucker Boulevard consisted of variably thick rubble fill overlying residual soils that give way to limestone bedrock. The rubble consisted of a mixture of clay with pieces of brick, concrete, mortar, slag, and glass—along with cinders, limestone, sand, and gravel—in varying proportions. The rubble was probably the waste product from the construction and demolition of the buildings along either side of the street. Rubble waste may also have been brought to the site from other locations and dumped beneath the bridge.

The natural soils at the site consist primarily of residual deposits comprising lean and fat clays, denoted as respectively CL and CH in ASTM International's standard D2487-11 (*Standard Practice for Classification of Soils for Engineering Purposes {Unified Soil Classification System}*). The soils exhibited standard penetration test *N* values ranging from 0 to 14 blows per foot. Moisture contents in the natural soil ranged from 18 to 49 percent, the higher values corresponding to the fat clays. Unconfined compression tests on the soil samples yielded undrained shear strengths ranging from 0.13 to 1.51 tons per square foot. Dry unit weights in the soils range from 79 to 108 pcf, the lowest values occurring in soft fat clays.

The limestone at the site was described as moderately hard to hard, moderately to slightly weathered, fine to medium crystalline, and thinly to thickly bedded. The depth to bedrock from the base of the excavation at track level ranged from about 8 to 17 ft. Most of the affected buildings along North Tucker Boulevard derive their foundation support from drilled piers or spread footings extended into the limestone. Groundwater was encountered at depths of 7 to 17 ft below the original railroad track level at the time of drilling in 2009.

The reconstruction of North Tucker Boulevard to original grades would require the placement of a maximum of about 30 ft of new fill immediately adjacent to buildings that had not been designed or constructed to resist lateral pressures from new fill. The primary geotechnical challenge therefore was to prevent the embankment fill from imposing lateral loads on the existing buildings and other structures that were to remain in place after the reconstruction.

To meet this design challenge and the prescribed design criteria, three alternatives for isolating the new fill from the buildings were investigated: walls of cast-in-place, reinforced concrete; mechanically stabilized earth walls with conventional fill; and EPS. The option of a reinforced-concrete wall that would be cast in place was rejected for several reasons. Among them was the fact that such a wall would have precluded long-term maintenance. Because the wall's outside concrete face would have been immediately adjacent to the existing buildings, access to the base of the wall would have been extremely limited or impossible in the future. Other disadvantages included the relatively long duration of construction, the relatively high construction cost, and bearing pressures sufficiently high to require protection of existing foundations, ground improvements, or deep foundation support.

Because local contractors had experience with it, the mechanically stabilized earth wall option offered the advantages of relatively low cost and reduced construction time. However, these benefits were overshadowed by certain disadvantages. Much like the cast-in-place concrete alternative, the mechanically stabilized earth wall option would have precluded or greatly inhibited future maintenance and would have entailed high bearing pressures, possibly requiring protection of existing foundations and ground improvements.

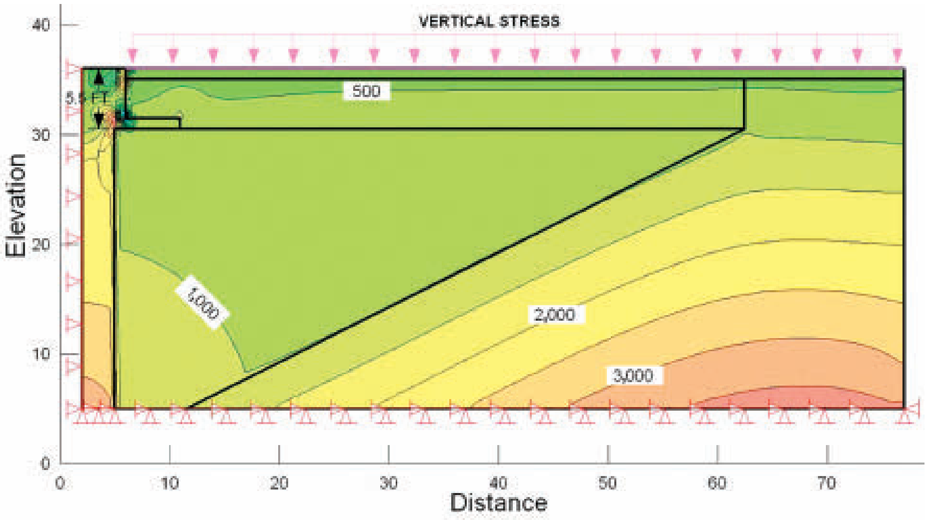
For its part, the EPS fill was found to offer several advantages. With its extremely light weight of 1 to 2 pcf, EPS is ideal for use in an embankment in which minimal consolidation settlement of existing soft clay foundation soils is desired. Easily shaped and manipulated to conform to irregular surfaces, EPS can be placed rapidly with the aid of small construction equipment or even by hand, requiring a crew of just two to four persons per block. Relatively strong and stiff, EPS can support light to moderate loading with minimal deformation and remain within the elastic range. With a Poisson ratio between 0.08 and 0.10, EPS would ensure that no lateral load transfer occurred between it and the adjacent buildings. The material would also damp vibrations from passing traffic and from seismic loadings acting upon the adjacent buildings. Finally, using EPS would significantly reduce the amount of fuel needed for excavating, transporting, and placing conventional earth fill. EPS was typically brought in by flatbed truck, each load delivering more than 100 cu yd.

With its extremely light weight of 1 to 2 pcf, EPS is ideal for use in an embankment in which minimal consolidation settlement of existing soft clay foundation soils is desired.

However, the EPS alternative did have some disadvantages. For example, the material is expensive, costing roughly $60 to $80 per cubic foot to install. Furthermore, local contractors were relatively unfamiliar with handling and placing EPS. The material can also be flammable and susceptible to degradation from hydrocarbon-based chemicals, including gasoline and diesel fuel. What is more, given its extremely low weight, EPS has a high potential for buoyancy under even a slight buildup of hydrostatic pressure.

Based on the findings of the alternatives evaluation, the design team devised a solution involving the placement of a combination of EPS and compacted soil fill within the irregularly shaped void beneath the street level. A 6 ft thick soil cap was included to accommodate the placement of utilities and to support tree growth along North Tucker Boulevard.

Analyses were performed to investigate the interaction between the EPS and the soil fill and to estimate the load deformation and stability of the proposed embankment



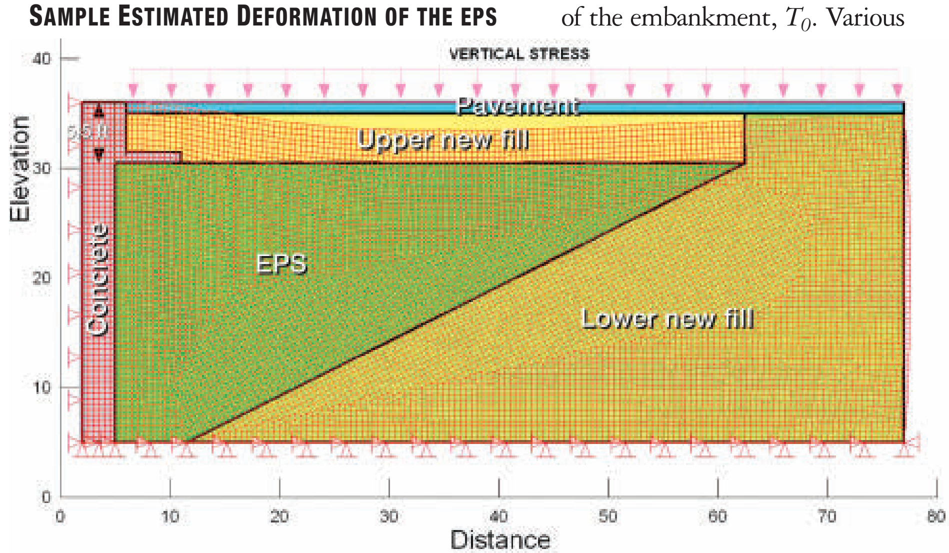
SAMPLE VERTICAL STRESS DISTRIBUTION THROUGH EPS AND CONVENTIONAL FILL AND LOAD TRANSFER TO EXISTING BUILDING

configuration when subjected to traffic and seismic loading. The program SIGMA/W—developed by GEO-SLOPE International Ltd., of Calgary, Alberta—was used to model the stress distribution and deformation of the EPS and to estimate the load transfer from the embankment to the building under traffic and seismic loadings. The design section consisted of a 6 ft thick soil planting layer, a 6 in. thick concrete load distribution slab, and about 25 ft of EPS fill extending down to native soils.

The figure on page 67 shows a typical design section of the EPS fill placed against the existing buildings and the conventional earth fill. A granular drainage layer was also placed along the interface of the EPS and the earth fill.

The SIGMA/W analyses were used to estimate the density of EPS fill required to resist the imposed stresses throughout various zones of the embankment. Finite-element analyses model linearly elastic properties, and because the linearly elastic properties of EPS, that is, the modulus of elasticity and the Poisson ratio, generally vary with density, several iterations were required to refine the model and converge on the required EPS density. Example output from the analysis showing the vertical stress distribution through the EPS and conventional fill, as well as the load transfer to an existing building, is shown in the figure above. Example output showing the estimated deformation of the EPS is shown in the figure at right. The visual representation of the deformation has been exaggerated by a factor of 100 to improve clarity. The maximum predicted vertical and horizontal deformations were estimated to total less than respectively 0.3 and 0.01 in. The results estimated minor, relatively uniform deformations within the EPS and across the soil section, indicating a low potential for differential settlement and permanent deformation of the EPS and pavement section.

SAMPLE ESTIMATED DEFORMATION OF THE EPS

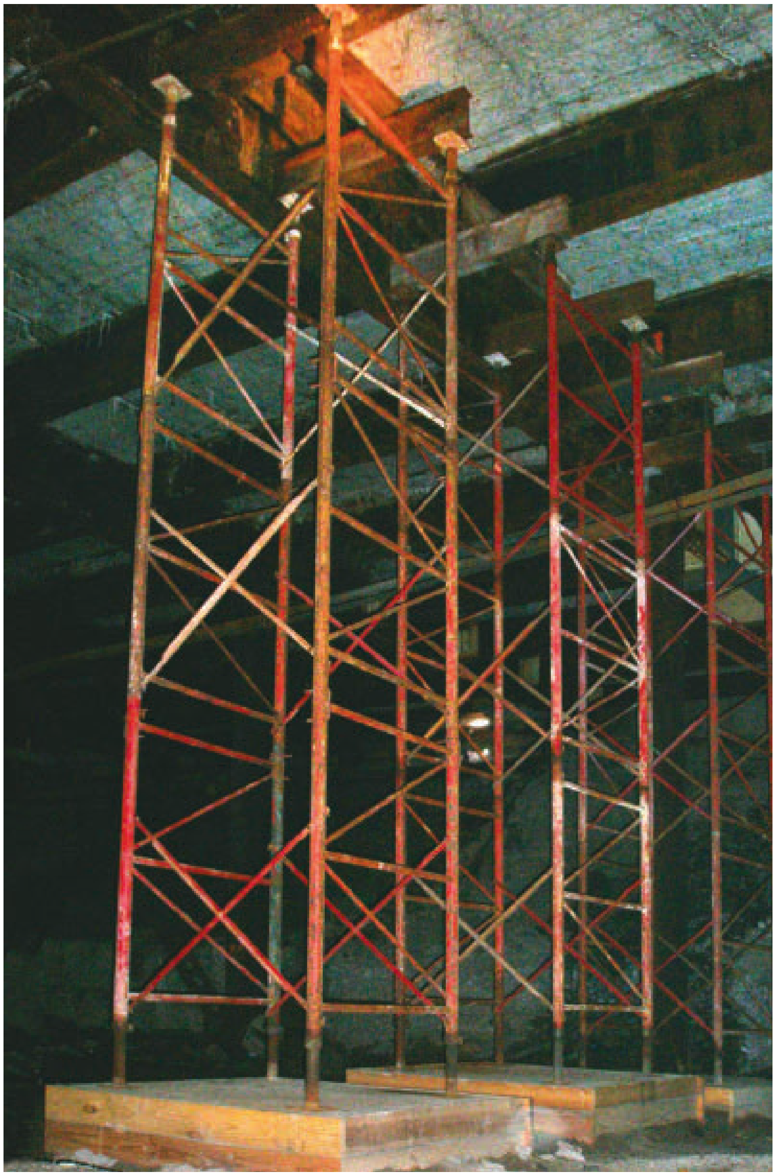


HDR, INC., BOTH

A rational approach to the numerical modeling for the seismic design and evaluation of freestanding EPS embankments was provided by Steven F. Bartlett, Ph.D., P.E., M.ASCE, an associate professor of civil and environmental engineering at the University of Utah, and Evert C. Lawton, Ph.D., a professor of civil and environmental engineering at the University of Utah, in a presentation at the 6th National Seismic Conference on Bridges and Highways, held in 2008 in Charleston, South Carolina. Bartlett and Lawton, along with two other researchers, presented additional findings on this topic in 2011 at the 4th International Conference on Geofoam Blocks in Construction Applications, which was held in Oslo, Norway. The principles they outlined were applied here in the seismic stability analyses. Two main sources were used in determining the design earthquake: the 2010 edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* (the acronym "LRFD" denoting load and resistance factor design) and ASCE's standard 7-05 (*Minimum Design Loads for Buildings and Other Structures*). The design spectral acceleration arrived at was for an event having a 1,000-year return period and a 7 percent probability of exceedance in 75 years. The peak ground acceleration associated with this event was estimated to be about 0.17*g*.

Global stability analyses of potential deep-seated failure surfaces under seismic inertial forces were performed using GEO-SLOPE International's SLOPE/W, a program that uses limit equilibrium techniques to search for the critical failure surface with the minimum factor of safety against instability.

The inertial acceleration for the embankment corresponded to the spectral horizontal acceleration value computed at the fundamental period of the embankment, T_0. Various



Before the reconstruction, the city used temporary shoring to support the failed ends of beams as a cost-effective way to keep the bridge open as long as possible.

HDR, INC., BOTH

values of T_0 were calculated along the alignment using methods discussed by Bartlett and Lawton and the methodology described in a February 2004 report entitled *A Technical Note re Calculating the Fundamental Period of an EPS-Block-Geofoam Embankment,* by John S. Horvath, Ph.D., P.E., a professor of civil engineering at Manhattan College. The values were calculated using the corresponding height, width, and elastic properties of the EPS embankment.

Ultimately, it was concluded that the highest embankment section of about 30 ft controlled the design because this section produced the highest inertial forces within the embankment and the lowest factors of safety against basal sliding. The corresponding T_0 value for the controlling section is about 0.92 second, and the associated horizontal acceleration is 0.18*g.* These values were used as the horizontal inertial acceleration in the limit equilibrium stability analyses.

The computed yield acceleration for the critical embankment section was estimated from the SLOPE/W analyses to be 0.17*g.* Therefore, Newmark sliding block displacement analyses were performed to estimate the permanent deformation for the design earthquake event. Using the computed acceleration at T_0 for the design event and the estimated yield acceleration, the estimated permanent deformation of the maximum height embankment section ranged from 1.7 to 1.8 in. To minimize the effect of the seismic load transfer, a synthetic compressible inclusion product was incorporated into the design between the embankment and the existing buildings. The synthetic inclusion provided predictable and consistent mechanical behavior that was useful in increasing the seismic stability of the embankment.

HDR submitted the feasibility study to the city in January 2009. Meanwhile, the project was selected to receive federal funding through the American Recovery and Reinvestment Act of 2009. As a result of the federal funding, the final design for the project was subject to a tight schedule that began in early April 2009 with the notice to proceed. Preliminary plans were due that July, while right-of-way plans had to be completed by September. Final approved plan documents had to be submitted in December 2009. The Missouri Department of Transportation allocated construction funds for the project in February 2010.

Because of the accelerated schedule associated with the federal funding, the project was divided into two phases. Properties included in the second phase had substantial right-of-way concerns that could not be addressed under the design schedule. This article has dealt with the first phase, for which construction began in March 2010 and was substantially completed by the fall of 2011. The second phase, which involved a separate design and was awarded to a different contractor, is still under construction.

The Gershenson Construction Co., Inc., of Eureka, Missouri, served as the contractor for the first phase, which involved several critical tasks, beginning with the relocation of affected utilities before the start of demolition. The existing steel bridge then had to be demolished and removed without affecting the adjacent buildings or impairing safety at the work site. The EPS blocks, which were provided by Versa-Tech, Inc., of Fredericktown, Missouri, were then shaped and placed to fit around irregular surfaces. The EPS material was placed simultaneously with the adjoining soil fill as the embankment elevation increased. The 6 in. thick concrete load distribution slab was then placed over the limits of the EPS fill. This slab distributes high point loads and protects the EPS from petroleum products. Next came placement of the 6 ft thick layer of additional soil fill above the concrete load distribution slab to facilitate tree plantings and make it possible to install utilities in the future. Finally, the

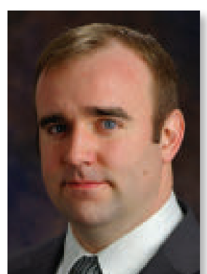
The EPS fill, used near existing buildings, offered several advantages in addition to its light weight. It can be shaped and manipulated to conform to irregular surfaces, can be placed rapidly by means of small construction equipment and small crews, and can support light to moderate loads with minimal deformations.

roadway base and pavement were placed. All told, the first phase cost $16.6 million.

Because it is a lightweight soil alternative, EPS is typically used when excessive settlement is anticipated. For this project, two factors supported the use of EPS. The first was the ability of vertically stacked EPS blocks to support loads immediately adjacent to the existing buildings without inducing lateral loads on the buildings. The second was the relatively maintenance-free nature of the EPS material over the long term.

Even though EPS and the techniques for installing it are relatively new to the construction industry, the project contractor quickly learned how to shape and place the material. The contractor praised the material, especially the fact that it could be placed quickly by a relatively small crew. On this project, the general limiting factor was how much material could be delivered to the site, as physical factors limited the number of trucks that could bring in material at one time. Of course, use of EPS will be limited by its high cost relative to conventional fill material, which, depending on density, typically costs just $^1/_6$ to $^1/_{10}$ as much as EPS. But as the North Tucker Boulevard project illustrates, in certain unique applications EPS provides a clear advantage over typical fill material and other structural systems. **CE**

Justin Anderson, P.E., M.ASCE, is a geotechnical engineer in the Lexington, Kentucky, office of HDR Engineering, Inc., which has its headquarters in Omaha, Nebraska. Patrick Poepsel, P.E., M.ASCE, is a senior geotechnical engineer in the firm's Omaha office, and Kevin Kriete, P.E., M.ASCE, is a senior bridge engineer in the St. Louis office.



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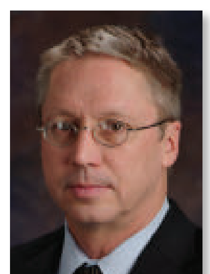

Poepsel

Kriete

This article is based on a paper presented at Geo-Congress 2013, a conference sponsored by ASCE and its Geo-Institute and held March 3–6 in San Diego.

PROJECT CREDITS **Owner:** St. Louis Board of Public Service **Designer:** HDR Engineering, Inc., Omaha, Nebraska **Contractor:** Gershenson Construction Co., Inc., Eureka, Missouri **Expanded polystyrene supplier:** Versa-Tech, Inc., Fredericktown, Missouri **Survey:** Engineering Design Source, Inc., Chesterfield, Missouri **Geotechnical investigation:** Geotechnology, Inc., St. Louis, and TSI Engineering, Inc., Kansas City, Missouri **Risk analysis:** ABS Consulting, Houston **Building investigation:** Pillar Design Group, Inc., St. Louis **Public relations:** Vector Communications, St. Louis