Design and Field Demonstration of ECC Link Slabs for Jointless Bridge Decks

Michael Lepech and Victor C. Li
University of Michigan, Ann Arbor, Michigan, USA

Abstract

The poor durability of concrete bridges throughout the US is an increasingly large concern for highway transportation authorities. With decreasing budget allocations for infrastructure maintenance, rehabilitation, and replacement, the need for greater durability is apparent. A main source of this poor durability is leaking mechanical expansion joints between adjacent simple spans of multi-span bridges. These joints typically require a large number of expensive maintenance or replacement projects over the service life of the bridge.

Using micromechanically designed ECC (Engineered Cementitious Composite) with a tensile ductility four hundred times that of normal concrete, expansion joints can be replaced by ECC link slabs, forming a jointless multi-span bridge. Utilizing the large tensile strain capacity and inherently tight microcracking properties of ECC, these link slabs maintain the simple span performance of the bridge while accommodating mechanical and environmental (i.e thermal) loads typically accounted for by the expansion joints. ECC link slabs allow for a joint free bridge deck, eliminating leaking problems which lead to low durability while creating a smoother riding surface. Design procedures and experimental testing of link slabs is reviewed and an ongoing field demonstration in conjunction with the Michigan Department of Transportation is discussed.

Keywords: ECC, bridge decks, durability, strain hardening, expansion joints, link slab

---

Michael Lepech
University of Michigan, Ann Arbor
1328 George G. Brown Laboratory
2350 Hayward Street
Ann Arbor, Michigan, USA 48109-2125

Email: mlepech@umich.edu
Tel: 734.883.4804

1.0 Introduction

As one of the most expensive public works projects in history, the contribution of the interstate highway system to the development of the United States as an economic power is doubtless. In 2002 alone, commercial freight transported over the interstate system accounted for 8.3 billion metric tons of shipments valued at over US$6.6 trillion [1]. The economic return on the initial investment of this expansive infrastructure system is immeasurable. However, it is essential that as the aging interstate system approaches the end of design service life, the transportation community remain committed to maintaining this critical economic lifeline. As recently as 1998, the American Society of Civil Engineers assigned grades of D- and C- to America's roads and bridges, respectively [2]. In 2002, the USDOT reported that over half of roads and bridges are in fair, mediocre, or poor condition, the majority of which are in heavily traveled urban areas [3]. The circumstances confronting transportation officials nationwide is serious.

One of the main durability and maintenance problems confronting departments of transportation nationwide are the continual failure of mechanical expansion joints installed between adjacent simple span bridge decks. While these expansion joints are essential to accommodate the large thermal deformations of the nearby decks, their tendency to quickly fall into disrepair and eventually leak is a constant source of deterioration of the entire superstructure. Water from the deck, saturated with de-icing salts during cold weather, leaks through deteriorated joints and ultimately corrodes the ends of steel girders, or penetrates into precast concrete girders and corrodes the reinforcing steel. Solutions to this continuing problem have been the development of continuous bridge decks or integral abutment bridges which seek to eliminate mechanical expansion joints by using an uninterrupted deck surface over multiple spans. However, these solutions are only applicable to new construction and present significant design complications within the superstructure or substructure when compared to simple bridge span design.

Recent research on Engineered Cementitious Composites (ECC), a type of High Performance Fiber Reinforced Cementitious Composite (HPFRCC), has shown them to be both highly durable and well suited for large infrastructure applications [4]. The primary reason for this high performance is the ability of ECC material to strain hardening under uniaxial tension while forming large numbers of microcracks up to a ultimate strain capacity typically over 4% as shown in figure 1. This large strain capacity is over 400 times that of normal concrete. However, unlike many other cement-based composites, this high level of tensile strain is not associated with large crack width openings. Typically, cracks within ECC material open to a maximum of between 50 μm to 70 μm during early strain hardening stages (i.e. below 1% tensile strain) and remain at that width under additional tensile strain up to failure (figure 1). These

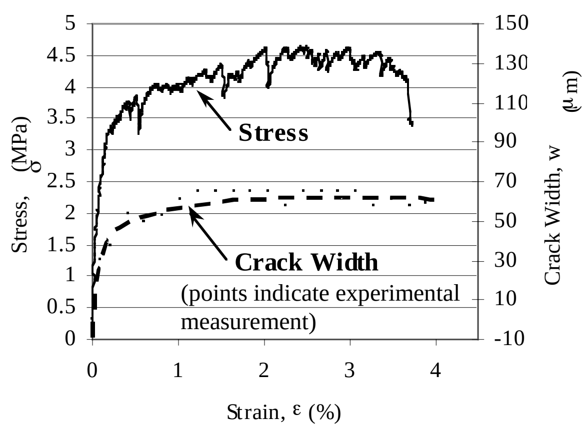

Figure 1. Stress-strain response of ECC material in uniaxial tension and development of crack width during straining.

unique characteristics can be attributed to deliberate micromechanical tailoring performed on the three phases within the composite; fiber, matrix, and fiber/matrix interface.

To allow designers to maintain simple span design assumptions, and allow for retrofitting of existing bridge structures, the use of ECC "link slabs", rather than mechanical expansion joints between adjacent bridge spans, has been proposed. By removing the expansion joint and replacing a portion of the two adjacent decks with section of ECC material overtop the joint, a continuous deck surface is constructed. The unique capability of ECC material to deform up to 4% strain in uniaxial tension while maintaining low crack widths allows the ECC link slab to accommodate the deformations imposed by the adjacent decks (i.e. due to thermal expansion and contraction) while protecting the underlying superstructure and substructure from corrosives present on the deck surface.

2.0 Design Procedures

2.1 Current Link Slab Design Practices and Procedures

To combat the continuing problem of deteriorating and leaking expansion joints, the Michigan Department of Transportation has recently constructed a number of concrete link slabs within Michigan. These link slabs are designed according to guidelines proposed by Zia et al [5] and Caner and Zia [6] in conjunction with the North Carolina Department of Transportation. These guidelines are based on previous research on theoretical analysis and laboratory experiments of simple span bridges (both steel and prestressed concrete girders) utilizing concrete link slabs to create jointless bridge decks.

Unlike ECC material, concrete link slabs do not possess the large tensile strain capacity and microcracking behavior and therefore must be heavily reinforced to keep crack widths within the concrete link slabs below acceptable serviceability limits allowed by the AASHTO bridge design code. This high reinforcement ratio within concrete link slabs unnecessarily stiffens the link slabs. Due to the inherently tight crack widths in ECC material, a high steel reinforcement ratio for crack control is not necessary allowing the ECC link slab to act as a hinge connecting the two adjacent spans and allowing for an easier design. An additional difficulty observed with construction of concrete link slabs is their sensitivity to poor construction practices. The large majority of concrete link slabs within Michigan which have shown distress or required maintenance were found to have too little reinforcement included in the design, or the reinforcement was not installed properly by the contractor [7]. This sensitivity to construction practices is of less concern with ECC since the performance of the link slab is dependent on the high strain capacity and low crack width of the material (i.e. material characteristic) rather than on the placement of reinforcement (i.e. structural characteristic).

2.2 Design Procedure of ECC Link Slab

To begin, the extents of the link slab are determined. The overall length of the link slab and the length of the link slab debond zone are calculated in equations 1 and 2, respectively. The debond zone is the center section of the link slab in which all shear connectors between the girder and deck are removed to prevent composite action between girder and deck (figure 2). Along with removal of shear connectors, a mechanical debonding mechanism is secured to the top flange of the girder to further prevent shear transfer between the girder and deck. This debonding mechanism may be either standard roofing paper (steel girder) or plastic sheeting (precast concrete girder), depending upon the type of girder. While composite action is retained in the adjacent spans, this debonding within the link slab allows it to function as a hinge between the two adjacent spans while they deflect. Zia et al [5] found that up to 5% of the adjacent deck may be debonded without affecting the simple span design assumption of the adjacent spans.

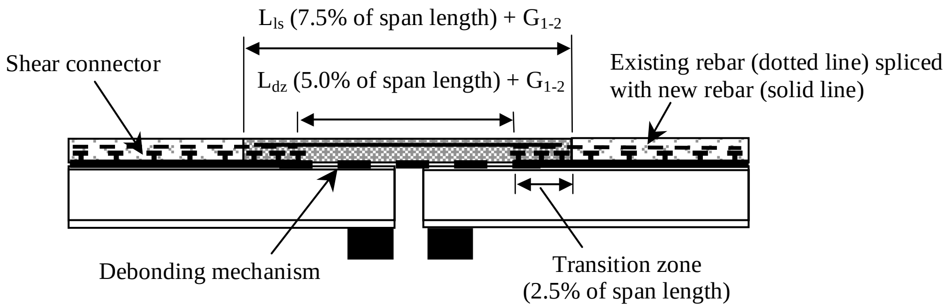

Figure 2. Schematic of ECC Link Slab

$$L_{ls} = 0.075 \cdot (L_1 + L_2) + G_{1-2} \tag{1}$$

$$L_{dz} = 0.05 \cdot (L_1 + L_2) + G_{1-2} \tag{2}$$

where L_{ls} is the overall length of the link slab in mm, L_1 and L_2 are the span lengths of the two adjacent bridge spans in mm, G_{1-2} is the length of any gap between the girders of the two adjacent spans in mm, and L_{dz} is the length of the link slab debond zone in mm.

Outside of the debond zone on either end of the link slab are the transition zones in which shear connection and composite action between girder and deck are reestablished. Due to the high shear stresses within the region, the number of shear connectors required by the design code is increased by 50%. The design of shear connectors in concrete according to the AASHTO design code has been shown conservative for shear connectors in ECC material. It is recommended to use the standard AASHTO design procedure for design of these shear connectors [8].

Following the calculation of link slab length, the maximum end rotation angles of the adjacent bridge spans due to live load must be determined through mechanics. This is a function of the maximum allowable deflection and the length of the adjacent spans as shown in equation 3.

$$\theta_{max} = \Delta_{max-short}\left(\frac{3}{L_{short}}\right) \tag{3}$$

where θ_{max} is the maximum end rotation angle of the adjacent bridge spans in radians, $\Delta_{max-short}$ is the maximum allowable deflection of the shorter of the two adjacent spans in mm, and L_{short} is the span length of the shorter of the two adjacent spans in mm. Since maximum allowable deflection is typically given as a function of span length (i.e. L/800), the maximum end rotation angle is often a constant for any span length. For instance, with Δ_{max} equal to L/800, θ_{max} will always be 0.00375 radians.

The uncracked moment of inertia is computed for the link slab per meter width of bridge deck using mechanics shown in equation 4.

$$I_{ls} = \frac{(1000\text{mm}) \cdot t_s^3}{12} \tag{4}$$

where I_{ls} is the moment of inertia of the link slab per meter width of bridge deck in mm^4, and t_s is the thickness of the bridge deck slab in mm.

Using the maximum end rotation of the adjacent bridge spans, and the moment of inertia of the link slab, the bending moment induced within the link slab per meter width of bridge deck due to the imposed rotations is calculated using equation 5.

$$M_{ls} = \frac{2 \cdot E_{ECC} \cdot I_{ls} \cdot 0.001}{L_{dz}} \theta_{max} \tag{5}$$

where M_{ls} is the moment induced into the link slab per meter width of bridge deck in kN-m, E_{ECC} is the elastic modulus of ECC material in GPa, I_{ls} is the uncracked moment of inertia of the link slab in mm^4 (equation 4), L_{dz} is the length of the link slab debond zone in mm (equation 2), and θ_{max} is the maximum end rotation angle of the adjacent spans in radians (equation 3). The elastic modulus of ECC material is typically assumed as 20 GPa.

With the imposed moment calculated, the amount of steel reinforcement within the link slab must be calculated to resist this moment. The amount of steel reinforcement within the link slab is based entirely on structural load capacity and not on any crack width serviceability requirements since large cracks do not form in ECC material under normal load conditions. To calculate the moment capacity of the ECC link slab section, a non-linear sectional analysis is used based on the assumption that ECC material is perfectly elastic-plastic. While ECC material typically does show some strain hardening characteristics after first cracking as shown in figure 1, this phenomenon will not be relied upon for conservative design practice.

The “yield strain” of the ECC material is set at 0.02%. From a pool of 40 separate tensile test results, this value is chosen as a fair representative for the first cracking strain of ECC material which will be used for the ECC link slab. The “yield stress” of the ECC material is chosen to be 3.45 MPa. While the actual ultimate strength is typically above this value, 3.45 MPa was again chosen as a fair representative value from the pool of tensile test results.

As proposed by Caner and Zia [5], a conservative working stress of 40% of the yield strength of the reinforcement is used for design. Unlike the design assumptions for concrete, in which no tensile force is carried by the concrete, a substantial stress of 3.45 MPa is assumed to be carried by the ECC up to failure between 3% and 4% strain. Using non-linear analysis and the assumption of a linear strain distribution within the section, shown in figure 3, the moment capacity of the section can be computed for any steel reinforcing

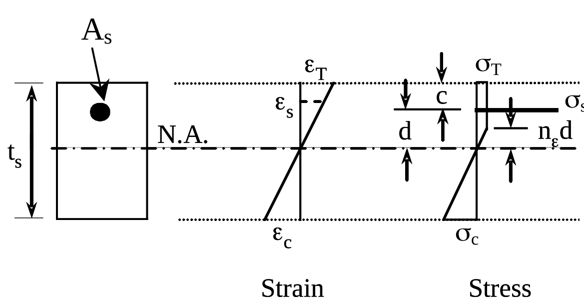

Figure 3. Stress and Strain Distributions in ECC link slab cross section

ratio. The reinforcement ratio is then adjusted accordingly to resist the moment due to maximum end rotation computed earlier in equation 5. Figure 3 also shows the cross sectional stress distribution of a reinforced ECC link slab (R/ECC).

To compute the moment capacity, the location of the neutral axis of the section must be determined. This is done through force equilibrium of the compression and tension portions of the section. However, prior to performing force equilibrium, the location of the "kink" in the tension region of the section, due to the elastic-plastic tensile response of ECC material, must be calculated. As a result of the linear strain assumption within the section, this can be done using geometry and knowing the ratio of the yield strains of steel and ECC, along with the assumption of 40% working stress in the reinforcing steel. This is shown in equation 6.

$$n_{\varepsilon} = \frac{\varepsilon_{y-ECC}}{0.4 \cdot \varepsilon_{y-steel}} \tag{6}$$

where n_{ε} is the yield strain ratio, $e_{y\text{-}ECC}$ is the "yield strain" of the elastic-plastic ECC behavior, and $e_{y\text{-}steel}$ is the yield strain of the reinforcing steel.

The equilibrium balance of the section is then calculated to determine the location of the neutral axis of the section. At this point a preliminary reinforcement ratio must be selected. A moment capacity for the link slab based on this reinforcement ratio will be determined and compared to the moment induced in the slab the beam end rotation (from equation 5). If the moment capacity for the selected reinforcement ratio is below the moment induced, a higher reinforcement ratio is chosen and another iteration of the design process is performed.

Equations 7a, 7b, 7c, and 7d are used to calculate the force within the reinforcing steel, tensile portion of ECC material, and compressive portion of ECC material per meter width of bridge deck. Equilibrium balance is completed by solving a simple non-linear equation, shown in equation 7e. The goal of this calculation is the determination of the value for "d".

$$T_{steel} = (0.4 \cdot f_{y-steel}) \cdot \rho \cdot t_s \tag{7a}$$

$$T_{ECC-1} = f_t^{'}\big((1 - n_{\varepsilon}) \cdot d + c\big) \tag{7b}$$

$$T_{ECC-2} = 0.5 \cdot f_t^{'} \cdot n_{\varepsilon} \cdot d \tag{7c}$$

$$C_{ECC} = 0.5 \cdot f_t^{'} \left(\frac{1}{n_{\varepsilon} \cdot d} \right) (t_s - d - c)^2 \tag{7d}$$

$$T_{steel} + T_{ECC} + C_{ECC} = 0 \tag{7e}$$

where T_{steel} is the tension force in the reinforcing steel per meter width of bridge deck in kN, $f_{y\text{-}steel}$ is the yield strength of the steel in MPa, ρ is the steel reinforcement ratio, t_s is the deck slab thickness in mm, $T_{ECC\text{-}1}$ is the tension force in the plastic ECC per meter width of bridge deck in kN, f'_t is the assumed tensile strength of the ECC material in MPa, n_{ε} is the yield strain ratio computed using equation 6, d is the distance from the neutral axis to the centroid of reinforcing steel in mm, c is the distance from the tensile face of the slab to the centroid of the reinforcing steel in mm, T_{ECC-2} is the tension force in the elastic ECC per meter width of bridge deck in kN, C_{ECC} is the compressive force in the ECC slab per meter width of bridge deck in kN.

With the force in each portion of the section known along with the location of the neutral axis, the moment resisting contribution of each portion can be used to compute the overall moment capacity of the link slab, as shown in equation 8.

$$M_{r-ls} = \left\{T_{steel} \cdot d + T_{ECC-1} \cdot \left(\frac{(1-n_{\varepsilon}) \cdot d + c}{2} + n_{\varepsilon} \cdot d\right) + T_{ECC-2} \cdot \left(\frac{2}{3}\right) \cdot n_{\varepsilon} \cdot d + C_{ECC} \cdot \left(\frac{2}{3}\right) \cdot (t_s - d - c)\right\} \cdot \left(\frac{1}{1000}\right) \tag{8}$$

where M_{r-ls} is the resisting moment provided by the link slab per meter width of bridge deck in kN-m.

With the moment resistance, M_{r-ls}, calculated from equation 8, this resistance is compared to the moment demand induced by the imposed end rotations, M_{ls}, from equation 5. If the resistance is greater than the demand, the strength design is completed using the selected reinforcement ratio. Otherwise, a higher reinforcement ratio is selected and the process repeated. Finally, a specific reinforcing steel bar is selected and the required bar spacing is calculated using equation 9.

$$s = \frac{A_{bar}}{\rho t_s} \tag{9}$$

where s is the spacing between the bars in millimeters, A_{bar} is the cross sectional area of the selected bar in mm^2, ρ is the finalized reinforcement ratio, and t_s is the deck slab thickness.

2.3 Material Considerations

To avoid failure of the link slab, the strain demand upon the ECC material both in tension and compression must be checked to ensure it does not exceed the capacity of the material. Once the location of the neutral axis is found, computing the strain at both the compression and tension face due to live loads on the adjacent spans is relatively simple using the assumed linear strain distribution. The strain in tension is computed using equations 10a and 10b, while the compressive strain is computed using equation 11. If these values computed in equations 10b or 11 exceed the tensile or compressive strain capacities of ECC material, a new version of ECC must be designed to meet these demands.

$$\varepsilon_{LL} = \frac{0.4 \cdot \varepsilon_{y-steel} \cdot (d + c)}{d} \tag{10a}$$

$$\varepsilon_{T} = \frac{\alpha_T \cdot \Delta T \cdot \beta L_{long}}{L_{dz}} + \varepsilon_{sh} + \varepsilon_{LL} \tag{10b}$$

$$\varepsilon_{C} = \frac{0.4 \cdot \varepsilon_{y-steel} \cdot (t_s - d - c)}{d} \tag{11}$$

where ε_{LL} is the tensile strain due to live load moment, $\varepsilon_{y\text{-}steel}$ is the yield strain of the reinforcing steel, d is the distance from the neutral axis to the centroid of reinforcing steel in mm, c is the distance from the tensile face of the slab to the centroid of the reinforcing steel in mm, ε_T is the maximum total tensile strain in the ECC link slab due to live load moment, shrinkage strains, and temperature deformations of adjacent spans, α_T is the coefficient of thermal expansion for girder material in 1/ºC, Δ_T is the seasonal temperature range in ºC, β is a design value taken as 2.0 for joints with two roller bearings and 1.0 for all other joints, L_{long} is the span length of the longer adjacent span in mm, L_{dz} is the length of the link slab debond zone in mm, ε_{sh} is the shrinkage strain of ECC taken as 0.001, and ε_c is the maximum compressive strain in the link slab.

2.4 Other Checks and Construction Sequencing

In addition to checking the ECC material capacity, the designer must perform a number of other checks. It must be verified that existing abutments can withstand additional thermal movement if all existing expansion joints are removed. If this is not the case, the existing backwall must be replaced with a sliding backwall. The designer must also verify that that the existing pier columns can withstand additional thermal movement if all existing expansion joints are removed. The existing bearings must be checked to verify they can accommodate additional thermal movements.

It must be noted that inherently assumed in this design example is a deck pour schedule which places the ECC link slab last, since the maximum end rotation of the link slab is calculated using only the maximum allowable deflection under live load (Δ_{max} = L/800). If the link slab is cast before all dead loads are applied to the adjacent spans, the combined dead load end rotation and live load end rotation may exceed the value calculated in equation 3. To this end, care must be taken during construction to place all dead loads on adjacent spans prior to ECC link slab casting.

3.0 Experimental Testing and Field Demonstration

Large scale laboratory testing of ECC link slabs was conducted to investigate the load capacity and fatigue performance of ECC link slabs, along with the development of cracking on the tensile face of the ECC link slab. For comparison purposes, a concrete link slab was also tested for load capacity, fatigue performance, and crack width development. This testing program is summarized by Kim et al [9] with results summarized in figure 4.

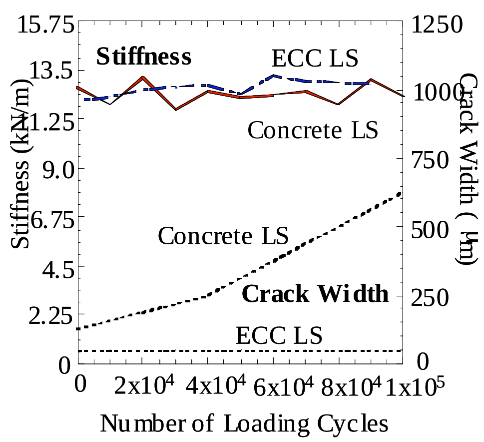

Figure 4. Stiffness and Crack Width Development of ECC and Concrete Link Slabs in Cyclic Loading

This study found that ECC material was a suitable choice for construction of link slabs to replace conventional mechanical expansion joints. The large tensile strain capacity, facilitated by saturated multiple cracking with widths of 60 μm meet all structural and durability needs of a link slab application. During monotonic loading, a lower stress in the reinforcement was seen in ECC link slabs than in concrete link slabs, allowing for further reduction of reinforcement levels. Cyclic tests revealed that both ECC and concrete link slabs show no degradation of stiffness after 100,000 loading cycles. However, crack widths in the concrete link slab grew to over 600 μm during cyclic testing while crack widths in the ECC link slab remained small, in all cases less than 60 μm. This allows ECC link slabs to better meet the durability and serviceability requirements and better protect underlying bridge superstructure and substructure.

A demonstration project, in cooperation with the Michigan Department of Transportation, is scheduled for construction during Summer 2005. This ECC link slab will measure 5.5m X 20.25m. Construction will comprise 25.5 m^3 of ECC, delivered on-site in a standard ready-mix concrete truck from a nearby batching plant. With an average daily traffic load of 10,000 vehicles per day, this will be the first large scale application of ECC material in North America.

4.0 Economic Considerations

Costing roughly \$100/m^3, Portland cement concrete is one of the most cost effective materials for use in infrastructure. However, this material has significant shortfalls, requiring intense maintenance and higher service life costs. High performance concretes, generally thought to have increased durability, or typical steel fiber reinforced concrete may cost up to \$200/m^3. The current cost of PVA-ECC material is approximately \$350/m^3. A commercial concrete patching material commonly used by the Michigan DOT is roughly \$540/m^3. While the cost of ECC may seem high in comparison to normal concrete, it is far less than many polymer concretes commonly used in repair applications, or certain high strength FRCs, which may cost \$2000/m^3-\$5000/m^3. With incorporation of industrial waste materials, lower cost and "greener" ECC materials are also undergoing development at the University of Michigan [10].

While material cost per cubic meter is of some concern to departments of transportation, the cost of the in-place material is of more consequence. With lower reinforcement ratios in ECC, substantial savings can be realized in lower cost of reinforcing steel and lower labor costs to install the reinforcement. Additionally, the self-consolidating nature of ECC, which requires no vibration after placement, allows for smaller crews potentially saving on placement labor costs.

Yet transportation planners must look into the future when selecting materials and designs which best meet infrastructure needs. Moderately higher initial investments in higher performing materials and technologies, such as ECC link slabs, may return substantial savings through the reduction of future maintenance and rehabilitation. These savings are realized both by the transportation agency and the motoring public. Current life-cycle research examining the use of ECC link slabs to replace mechanical expansion joints has shown that the ECC system has lower life-cycle costs than the conventional joint system. The difference between the two systems represented a 15% cost advantage for the ECC system over the 60 year life-cycle considered [11].

5.0 Conclusion

Without doubt, transportation officials are facing an increasing crisis when dealing with the rising cost of maintaining an expansive infrastructure system coupled with ever shrinking funds. One solution is the use of new materials and design technologies which look to increase the level of performance, improve durability, and extend the service life of infrastructure systems. One such example is the introduction of ECC link slabs to replace mechanical expansion joints between adjacent simple bridge spans. With the unique mechanical characteristic of strain hardening under uniaxial tension up to 4% strain capacity while forming small closely spaced microcracks, ECC material meets all requirements essential for the link slab application. The necessity of allowing for the deformation of adjacent bridge spans under thermal loads, along with creating a uninterrupted deck to protect the underlying superstructure and substructure is met.

Using the procedure outlined above, the design of an ECC link slab can be completed for any current or planned multi-span bridge designed using simple spans. Large scale tests confirm that ECC link slabs perform as desired, and substantially better than concrete link slabs. Finally, the cost advantages associated with ECC far outweigh the higher initial material cost. When considering the savings in steel reinforcement and labor, the use of ECC in many infrastructure applications becomes economically feasible. When considering life-cycle costs which include maintenance and rehabilitation of deteriorated structures, using ECC in infrastructure applications, such as the ECC link slab, becomes not only feasible, but advantageous.

6.0 Acknowledgements

The authors would like to graciously thank the Michigan DOT and the US National Science Foundation MUSES Grant (CMS-0223971 and CMS-0329416) for partially funding this research, in particular Mr. Roger Till, P.E. and Mr. David Juntunen, P.E of MDOT. The authors would also like to thank Dr. Yun Yong Kim for his advice and discussions.

7.0 References

1. USDOT - FHWA, *Highway Statistics 2002*. Washington, D.C., USA, 2003.
2. American Society of Civil Engineers. "*1998 Report Card for America's Infrastructure*". March 5, 1998. http://www.pubs.asce.org/news/sep3.html. Accessed December 7, 2004.
3. US DOT, Bureau of Trans. Statistics and US Dept. of Commerce, U.S. Census Bureau, 2002 *Economic Census*, *Transportation Commodity Flow Survey*. Washington, D.C., USA, 2003.
4. Li, V.C. 2003 "On Engineered Cementitious Composites (ECC) – A Review of the Material and its Applications," *Journal of Advanced Concrete Technology*, 1(3), pp. 215-230
5. Zia, P., A. Caner, and El-Safte, A.K., 1995, Jointless Bridge Decks, Research Project 23241-94-4, Center for Trans. Engin.Studies, North Carolina State, pp. 1-117
6. Caner, A. and P. Zia, 1998, Behavior and Design of Link Slabs for Jointless Bridge Decks, *PCI Journal*, May-June, pp. 68-80
7. Gilani, A. and Jansson, P. "Link Slabs for Simply Supported Bridges - Michigan Department of Transportation Report No. MDOT SPR-54181." Michigan DOT. Lansing, MI. 2004.
8. Li, V.C., G. Fischer, Y. Kim, M. Lepech, S. Qian, M. Weimann, and S. Wang, 2003, "Durable Link Slabs for Jointless Bridge Decks Based on Strain-Hardening Cementitious Composites." CEE Department, University of Michigan, pp. 1-96
9. Kim, Y.Y., Fischer, G., and V.C. Li. 2004 "Performance of Bridge Deck Link Slabs Designed with Ductile ECC," *ACI Structural J*., 101(6) pp 792-801.
10. Li, V.C., M. Lepech, S. Wang, M. Weimann, and G. Keoleian, "Development of Green ECC for Sustainable Infrastructure Systems." in *Proc. Int'l Workshop on Sustainable Development and Concrete Technology*, Beijing, China, Ed. K. Wang, 181-192, 2004.
11. G. Keoleian, A. Kendall, R. Chandler, G. Helfand, M. Lepech, V.C. Li, "Life Cycle Model for Evaluating the Sustainability of Bridge Decks", to appear in *Proc. 4th International Workshop on Life-Cycle Cost Analysis and Design of Civil Infrastructure Systems*, Florida, May 8-11, 2005