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Review Paper on Link Slab Bridge Girder Technique

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***Abstract*— Numerous transportation agencies in Canada and the United States have studied alternatives to expansion joints in bridges because of high maintenance costs and tacky joint durability. One of the alternatives is the used of link slab in a jointless bridge, which links the adjacent bridge deck slabs at the pier, producing a continuous slab over the bridge spans. While the link slab technique can present the benefits of a continuous bridge deck, improvement of the design and detailing of the link slab itself is required to optimize this bridge deck system and assure long-term performance. Materials with high tensile strain capacity, such as fiber reinforced concrete (Engineered Cementitious Composite), may be utilized for application in the link slab to enhance the strength, durability, and cracking characteristics of the link slab. The main objective of the paper is to outline the history of Link slabs that represent a new technique to overcome the drawbacks of expansion joints. By experimental tests and numerical models.**

Keywords—Link Slab; Expansion joints; Engineered Cementitious Composite.

I. INTRODUCTION

A common practice in bridge building since the 1960s is to build multiple span bridges as simply supported girders with expansion joints at the support locations. These expansion joints simplify design and give space to accommodate different movements in the deck as well as other short and long-term displacements induced by vehicular traffic, shrinkage, thermal variation, creep, and other loads [1] and [2] However, the existence of the mechanical joints leads to extra maintenance costs, rider discomfort, and later could become a cause of deterioration as moisture and the leakage of contaminated water producing the deterioration of the supporting substructures through early corrosion. The United States Federal Highway Administration (FHWA) and specific state departments of transportation (DOTs) are actively enhancing the establishing of jointless bridges, i.e., bridges without expansion joints, to treat the durability problems and poor deck joints performance. Jointless bridges have been utilized in numerous states.

Two solutions to eliminate bridge deck joints have been selected in the US, specifically an integral bridge method with girder continuity and a jointless bridge deck method with simply supported girders. [3] Observed that the jointless deck system application is usually more efficient than the integral bridge system application. Most problems related to deck joints can be removed by utilizing continuous deck systems while giving the girders in the system with a degree of continuity [4]. The removal of deck joints produces additional forces that take into consideration during the design of the bridge systems. Bridge deck joints enable be substituted at the abutments by utilizing an integral abutment, semi-integral abutment, or deck extension. At piers, joint removal can be performed by utilizing continuous for live load bridge structures or by installing link slabs.

Link slabs are typically utilized for substitute deck expansion joints above piers in short and medium multi-span bridges. A link slab is the part of the deck over the joint that linked two adjacent deck spans, producing the deck as a fully continuous method while the girders stay simply-supported [5]. Link Slabs can be performed in prestressed concrete and steel girder bridge superstructure. The major purpose of creating the deck continuous is to avert span separation and deterioration caused by leaking [4] and [6]. Figure. 1 presents the replacement of deck joint approach utilizing link slabs. This study focuses on the link slab approach for bridge integration, which is less intrusive and hence has a better potential of gaining acceptance on a large scale. The integration is achieved by constructing new bridges with jointless decks or redecking expansion joints in repair and maintenance work of existing bridges.

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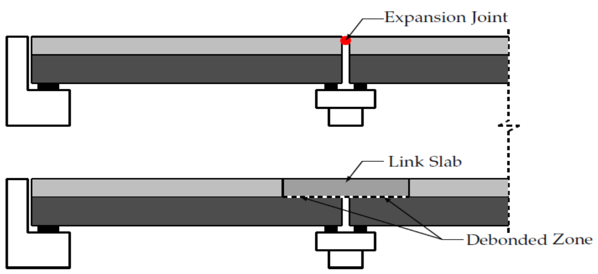

Figure 1: Deck joint replacement approach with link slabs

II. LITERATURE REVIEW

In [7] provided an overview of jointless bridge decks over continuous girders in Tennessee DOT. Tennessee policy reads, the bridges built must be continuous from end to end with no intermediate joints except for cold joints needed for construction and apply both longitudinal and transverse joints. [8] In his case study confirm that ridding an existing bridge of expansion joint does not need complex analysis and design methods. The installation sequence for A.L. Blades and Sons of Hornell, N.Y. is presented. [9] Presented the knowns and unknowns of the jointless bridges. They define the real behavior of a jointless bridge is a greatly complex due to the generating of secondary forces cause of temperature effects, creep, and shrinkage. Notwithstanding these complexities, jointless bridges execute satisfactorily.

[10] Suggested state-of-the-art techniques to analyze the jointless bridges for primary and secondary loads. They believed in five in-service jointless bridges for their analysis. In the second study effort, [11] suggested design alternations in a jointless bridge based on conclusions from the experimental and analytical results. [12] Briefly explains the details of simplified continuity for short and medium-span composite steel girder bridges in the state of Tennessee. He else displays the details of the link slab, continuity for composite load details, and continuity for dead load slab and composite load details.

[13] Performed an analytical study of the load-deflection response of this jointless bridge deck system by using a finite element solution. A numerical solution was discussed in his research for the study of the jointless bridge deck. The girders were preserved simply supported while the concrete deck was continuous. The analytical methods to investigate the elastic and inelastic behavior and strength of jointless bridge decks were studied. The numerical method included isoparametric beam elements for representing the girders and deck's cross-sectional and material properties. The full-depth link slab was represented by a uniaxial spring element located at the deck's centroid line which had the same length as the spacing between the adjacent girders' ends.

[14] Executed a simplified design method for the removal of expansion joints for bridges utilizing partially deboned with continuous decks. This model was studied utilizing linear analysis and design for a jointless bridge deck on the simply supported steel girders.

[15] Updated the work performed by Gastal [13] to calculate for the partial debonding length of the deck from the end of the girder. It had been taken into consideration that the length of partial debonding (Link Slab) longer than the distance between adjacent girders through breaking the composite action between the bridge deck and the girder for a specified distance. The length of the debonded zone was ranged from 1% to 8% of the length for each adjacent spans. In this work considered simplified computational methods by modeling the girders and deck by two-noded isoparametric beam element, and link slab by two-noded uniaxial spring-like element with only axial stiffness.

[16] Worked on constructing full link slab details for the Department of Transportation in North Carolina continuous bridge deck. They decided to perform their bridge deck continuous with simple span girders. The portion of the deck joining two simple span girders adjacent ends is mentioned to as the link slab. [16] Were one of the leading research groups to develop methods for designing the link slab. The steps of research were:

- Laboratory research on scaled prototypes of a link slab bridges was performed and has been studied deflection, rotation of the link slab and the force effect.
- Computational models were improved (i.e., structural analysis program) to expect the force effects, deflection, and rotation of the link slab bridge.
- Equations were obtained for the force effects on the link slab bridge and results were compared to the analytical model and empirical results.

[5] Have studied link slab experimentally to give recommendation on the design of link slab. The configuration of the author's bridge under study was continuous deck with simple span two girders. The link slab was deboned from each girder for a distance of 5% (26 in) of the girder length, see Figure. 2. This was achieved by omitting girder stirrups at this zone and by placing two layers of plastic sheets between the deck and the girder. In order to reduce stresses in link slab and consequently the cracks in its mid-zone, the concept of the debonded length between girder and deck was introduced [5]. Adjusted [16] 2D finite element models were built to model this zone and to study the effect of different support conditions on the induced stresses in link slab [11].

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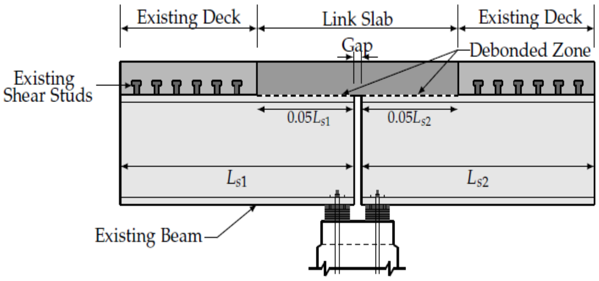

Figure 2: Debonded link slab configuration tested .Source [5].

Caner and Zia [5] observed that previous analytical methods developed by [15] and El-Safty [16] unsuccessful in the forecast the behavior of the jointless bridge decks. The conflict between the results of the experimental and model forecast was attributed to forming the link slab element that represented only with axial stiffness. Based on the results of the study, the researchers implemented techniques for the analysis and design of link slabs. The analysis program was utilized to calculate stresses in the steel bars and predictions of surface crack width in the middle of the link slab. All elements in the bridge model, including the link slabs, were represented by conventional beam elements. The program was verified utilizing the experimental results.

Caner and Zia [5] have suggested the initial formal design procedure for link slabs based on the experimental study represented in equation 1. This method is currently often utilized by DOTs in the United States. The method idealizes the link slabs as pin connections. It implies that link slabs have low stiffness compared to that of the girder's cross-section, which leads to negligible continuity between the bridge systems.

$$M_0 = 2E_c I_d \theta / L \qquad \text{equation 1}$$

I_d (the moment of inertia of the link slab). Θ (Girder end rotation of girder. Ec (modulus of elasticity of link slab) L (Girder Span Length).

Caner and Zia [5] have investigated that the girders can be designed as simple-span beams because of the negligible continuity of link slab. Link slabs can substitute the interior expansion joints in bridges of up to four spans. The link slabs should continue to be deboned from the deck for 5% of the length of the girder supports. They also proposed that saw cuts be made at the middle part of each link slab to improve control cracking, and epoxy coated reinforcement or non-metallic reinforcement be used to decrease the risk of corrosion.

These investigations showed that the link slab was subjected to bending under normal traffic conditions instead of axial elongation (act as beam instead of tension member). Because of the negative bending moment, generated tensile cracks on the surface of the link slab, that's under service conditions. They noted that there are other factors that contribute to the occurrence of cracks on the surface of the link slab, that's because of exposure the link slab to additional stresses due to creep, shrinkage, and temperature loading, and should be monitoring the crack width.

In [17] studied the use of link slabs as a seismic retrofit system for present multi-simple-span highway overpasses. The researcher conducted the case study on simply supported spans a 90-m long bridge with two traffic lanes. AASHTO Type III Bridge girders were that was supported on same elastomeric bearing pads at all ends. They concluded that the connection given by the link slabs can eliminate the span separation obstacle and possible damage due to removing. The proposed design method can be used for the preliminary design of link slabs.

In [18] they decided to observe and evaluate the performance of the bridge by using remote instrumentation and other analysis methods. This study considers the first illustration of a link slab established in North Carolina (Figure.3). The design procedure generated by Caner and Zia [5] was utilized for the design of the link slabs. They have not presented the configuration of bearing and the debonded technique between girder and deck. The bridge was divided in half, and only one side bridge was instrumented and observed. Therefore, the first span had an integral abutment at the beginning of its span, and the second span was free at the opposite end. The debonding length for the link slabs was 5% of the girder length, and it was designed treating each span as a simple-span. The girder end rotations and temperature variations were registered during the remote instrumentation. A full scale live load test was also conducted. One of the principal theories to be confirmed that the bridge girders can be designed as simple-span beams for dead and live loads.

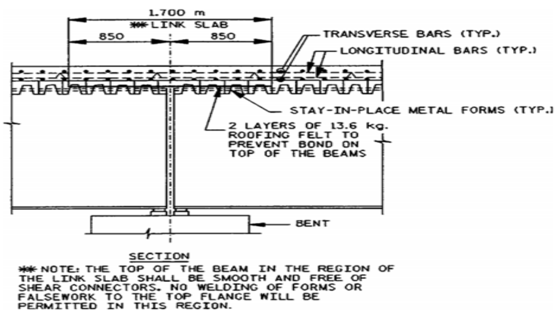

Figure 3: Link slab detail. Source [18].

The live load test was conducted at four load levels at two different positions. The positions provided the maximum positive and negative moment in the link slab, respectively. The loads were selected to the empty truck, maximum permissible load without a permit, midway between the empty truck, maximum allowable load without a permit, and the maximum load admitted with permit. The rotations of the girders were measured. The measured temperature produced rotations were determined and. Rotations due to service load were neglected compared to rotations due to thermal. The girder end rotation induced by the thermal was smaller than predicted. Some of the conflicts can be assigned to how and under what circumstances the monitoring equipment recorded.

All measured rotations through the year and through the live load test never reached the design rotation of 0.002 radians used for the link slab. Controlling the link slab cracks occured by using the saw cut. The cracks formed in the link slab surface with the width of crack 0.063 inches (1.6 mm) but did not change the width. Crack width is 0.013 inches (0.33 mm) which is wider than the AASHTO allowable limit for design. The crack was assumed to have been caused by localized debonding of the concrete.

In [19] proposed the effects of the support configuration on the link slab system. The designers developed simplified techniques for the jointless bridge decks by flexural analysis, which includes the classic three-moment equation. Also, in [19] prepared a simplified method for the analysis of bridges with jointless decks based on Caner and Zia [5] link slab for use by bridge designers. A parametric study is a behavior for the case of a two-span bridge. They dealt with link slab bridges as partially continuous systems because of the unequal girder end rotation on each side of the link slab. They used two support configuration in their analyses – HRRH and RHHR. The hinge supports restrict longitudinal movement, and the rollers permit longitudinal movement.

These support configurations were chosen because they agreed to the upper and lower limits of the behavior, sequentially. In the deriving equations, it was supposed that link slabs only provide axial stiffness neglecting any flexural rigidity they may contribute. The suggested equations were verified using laboratory data from Caner and Zia [5]. From the comparison, Okeil and El-Safty [19] stated that inconsistency between the deflection of experimental results and the theoretical results for the RHHR configuration. The researchers demonstrated that this difference was because of support movements that would prevent the production of large continuity moments [19] and [20]. A two-dimensional (2D) finite element model was created to check their condition. The bridge model has consisted of a two-span bridge and deck thickness was 7.5 in. The web of steel girders and the reinforced concrete deck was modeled as a plane element. On the other hand, the flange of girder and link slab was modeled as the truss element.

The support configuration – HRRH and RHHR – concludes which equations and factors should be used. The factors used in the analysis are the ratio of span length, link slab stiffness coefficient, variable axial rigidity, and shape factor. They calculated the continuity moment and tensile axial force for the link slab at a 1% reinforcement ratio. The selected range of these variables was then used to provide the design charts and tables.

In [19] performed a parametric study on a two-span link slab bridge to evaluate the effects of parameters such as girder types, support conditions, steel reinforcement ratios, and loading arrangements. The analytical results displayed that the stiffness and tension force generated in the link slab was influenced by support configuration. A debonded length ranged between 2% and 6% of the girder span, rely on the support configuration and loading case, which was established to be sufficient to obtain the maximum load capacity in the link slab. The increase of the steel reinforcement proportion in the link slab to an acceptable limit.

In [19] showed an example on how to use the design charts. They concluded that the tension force that develops in the link slab is affected by the different support configuration. They also derived expressions for the tension force and continuity moment. They concluded the form of the hinge support to lead a higher continuity moment and tension force in link slab. In the case of HRRH, the girders are permitted to move nearby to each other than in the RHHR case, and the tensile axial force and also the continuity axial force in the HRRH case is lower in value than the RHHR case. Figure.4 (a) illustrate the deformations in the periphery of the interior support for the RHHR support configuration. Deformation of girder and end rotations lead to move the deck apart because of applied loading. On the other hand, Figure.4 (b) showed that at HRRH support condition the deformation of the girder and end rotations the surface close due to the applied loading.

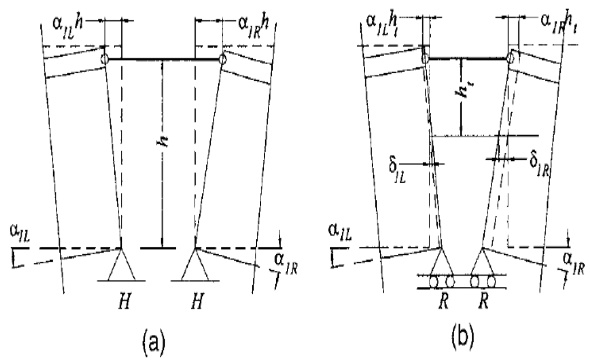

Figure 4: Force method for RHHR and HRRH Link Slab Bridge adopted from [19].

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In [21] suggested details to solve the interface cracking problem of traditional link slabs, that's due to high-stress concentrations among the interface of the link slab and the existing concrete deck. Shown in Figure.5, the link slab suggested was divided into two zones the first zone called deboned zone that represents 5% of the adjacent span girder and another length represent 2.5% of the adjacent span girder was named transition zones. This transition zone act to transfer the interface stresses away from the debonding start point. Observed that the transition zone included shear studs from the end of link slab form each side. Thus, composite action was created between the link slab and girder. In addition to the adjusted design detail, Also, in [21] proposed the used of engineering cementitious composite (ECC) for link slabs.

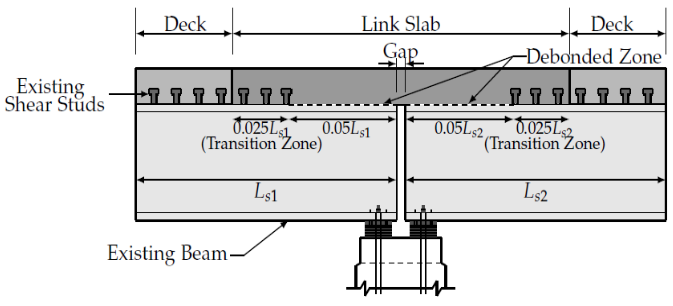

Figure 5: ECC link slab configuration proposed by [21].

In an effort to develop on the traditional reinforced concrete (R/C) link slab design, they suggested in [22] and executed a link slab design in a full-scale field using a highly ductile Engineered Cementitious Composite (ECC). It was reasonable to decrease some of the steel reinforcement for controlling crack width, needed by the transportation authorities, due to the essentially tight crack widths in ECC. Consequently, because reinforcement ratio is low, the ECC link slab was behaving like a hinge compared to the stiffer traditional concrete link slab. The ECC link slab was designed according to the American Association of State and Highway Transportation Officials (AASHTO).

Engineered cementitious composite (ECC) is a unique type of High Performance Fiber Reinforced Cementitious Composite (HPFRCC) that allows the great potential to resolve the durability performance of reinforced and pre-stressed concrete structures [23,24 and 25]. The high strain hardening characteristic and multi micro-cracking behaviors undergo tension and flexure while reducing the amount of the reinforcing fibers (less than 2% by volume) makes it a perfect material for the link slab implementation [23 and 26].

Previous tests proposed that the bond strength between concrete and ECC (hot joint: both materials are poured at the same time) is approximately 0.3 ksi by [27]. On the other hand, in case of bridge rehabilitation, the interface is cold jointed when an ECC link slab is utilized for the replacement of an expansion joint, indicating that the bond strength can be much lower than that of a hot joint. Techniques to improve the interface between concrete and ECC and design ways to decrease stress concentrations at the interface are of main concern for interface design by [23].

Li et al. [23] studied that the current design method for concrete link slabs does not give sufficient attention to the design interface between the deck and link slab. In traditional concrete link slab design, extra reinforcement is spliced with the existent reinforcement to strengthen the link slab. Nevertheless, these reinforcements usually end at the interface. Also, the debond zone (part of the link slab is designed debonded from the girder to provide more flexibility to the link slab) starts at the interface.

Li et al. [28] observed that reinforcement stress is not an appropriate criteria to discover the required reinforcement ratio in the link slabs. The designers stated that utilize of an uncracked section to get the applied moment on link slab. The expected longitudinal amount of steel reinforcement was overestimated that's for satisfying limit stress criteria. Accordingly, the design method by Li et al. [28] determines the amount of steel reinforcement in the link slab, which depends on the section capacity moment. A non-linear sectional analysis is utilized to determine section capacity moment assuming that the behavior of ECC material was perfectly elastic-plastic during the service. In addition, designers have utilized the conservative working stress of 40% of the yield stress of the steel reinforcement for link slab design.

In [29] studied using the minimum reinforcement ratio. They performed many iterative trials to get adequate steel reinforcement ratio. During the analysis, the chosen reinforcement ratio was modified to resist the applied moment imposed by the girders end-rotation. Different from traditional RC link slab, the capacity of structural loading used to resist moment instead of serviceability because of the micro-cracking behavior of ECC.

In the following study [29] presented the field illustration of the improved design detail displayed in Fig. 5 in a bridge in Michigan. Cracking was detected after three days of placement and before curing was finished. Several cracks formed within the link slab. Cracks width were 0.005 in. to 0.014 in and spacing around 8 in. they start from the steel bars and propagates towards the surface. Several severe cracks were detected around the steel rebar which stands out surface of the link slab to be cast into the sidewalk. The cracking referred to gradients of the temperature, shrinkage stresses, or loss of much water at the early ages.

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In [30] authors found that causes of the early age cracking maybe below water-to-cement ratio, low-retarder-to-cement ratio, utilization gravity-based mixer, wind effect, low curing relative humidity, high curing temperature, and skew angle. It was recommended to limite skew angle to 25, cast at the night time, and water treatment for at least 7 days.

Link slab moment capacity was calculated by the nonlinear analysis with assuming that the response of ECC is perfectly elastic plastic. For the reservation design practice, the linear tensile strength of the ECC was used while neglecting the strain hardening characteristic (as seen in the typical stress-strain properties, Figure. 6. After the first cracking Lepech and Li [29]. After comprehensive results of the tensile test on ECC, Lepech and Li [29] [31] recommended yielding stress and strain of 3.45 MPa and 0.02%, respectively based on the first cracking responses

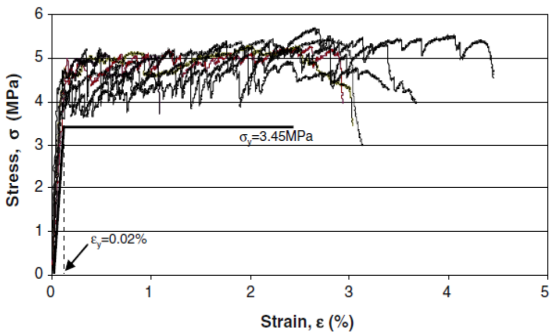

Figure.6: Idealized Elastic-Plastic Response of the ECC [22].

In [32] the link slab behavior and partial continuity with jointless decks for various support conditions and also skew bridge angle were studied. It was confirmed that:

1-The partial continuity develops as a result of the axial stiffness of the link slab rather than its flexural rigidity, which leads to larger average stresses, smaller extreme fiber stresses in the link slab

2- The continuity moment in the link slab is reduced with the increase in the ratio of debonding length. Similarly at increased bearing stiffness.

Though the utility of link slab, the bridge deck durability and performance were improved to some extent, however, large tensile cracks appeared on the link slab part for the traditional normal concrete [5]. The formed cracks permit salt water to reach reinforcement and girder producing rebar corrosion and finally leading to structural failure. That’s similarly to the mechanical expansion joint.

In [33] conducted a field investigation of eight bridges in Michigan to define the link slabs performance and other components of the bridge such as sleeper slabs, approach slabs, backwalls, and bearings configuration. All the investigated link slabs were designed by [5] procedure.

Note that the reinforcement's lower layer is discontinuous. The major problems mentioned in the survey showed that the existence of the transverse crack with full depth and wide width at the centerline of the link slab. The cracking was attributed to poor construction practices including insufficient placement of the longitudinal steel, and other factors.

In [33] implemented a 3D finite element model analysis to study factors influencing jointless bridge deck cracking. The span of each girder 834 in and full thickness for the deck and link slab was 9 in. Link slabs were designed with the bottom layer of reinforcement was discontinues over the centerline of the piers. The design parameters studied were support configuration, length of debonded zone, hight of the girder, and ratios of the adjacent span length. Three various support conditions were studied: HRRR, RHHR, and RRHR, length of debonded zone was changed as 0%, 2.5%, 5.0%, and 7.5% of the span length. The two standard AASHTO PCI sections studied were Type III and Type VI. The total length of the link slab was also increased from 85 into 98 in. The models were undergoing both live HL-93 and thermal loads, as given by AASHTO specifications [2017].

Based on the results of the analysis, it was found that cracks were formed on the link slabs because of thermal hydration and stresses of the drying shrinkage. It was noted that support configuration greatly affected on the link slab behavior. With RHHR support condition the link slab subjected to a moment and axial force. The analysis also exhibited that drying and thermal hydration shrinkage strains can form cracks of which the width coincides with the supposed under a live load. The authors suggested including the added moment and axial force produced from gradient thermal loads to the design of the link slab.

A new approach for the analysis of link slabs was suggested that involved the utilization of an axial load versus bending moment interaction diagram to those certain support conditions in which the link slabs were exposed to coupled flexural moment and axial loads. The research recommended prevailing cracking through the improvement of new cementitious materials. It was also recommended the utilize continuities bottom and top meshes reinforcement that's for the different support conditions. As well as a saw cuts over the centerline of the pier and at either side of the link slab to monitor cracking [34].

In. [33] suggested a design approach based on the bridge analysis models with zero skew angle and exposed to live load and thermal loads in [35] modified the procedure to combine the effects of the skew angle on the moment of the link slab. The investigation involved finite element modeling of skewed link slabs subjected to a static live load and gradient thermal loads as well as field evaluation of a skewed bridge. Also, they studied effect of the different bearing conditions on the behavior of the link slab. Design suggestions were developed for the use of high skew bridges with link slab. The study concluded that the moment formed in a link slab under thermal gradient loads stayed the constant irrespective length of the span. It was noted that the moment evolved in a link slab under live load decreased with an increased length of the span.

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The researchers observed the minimum amount of required reinforcement based on AASHTO specifications to be sufficient for skew link slabs at specific support condition (i.e., HRRR or RRHR). Proposed additional reinforcement bottom layer to resist the high tensile stresses generated near ends of the debonded region. It was also proposed the construction joint at the partially debonded and fully bonded zones. Deflections in the bridge and translations that occurred at the support were measured under both live and thermal loads.

In [36] submitted details of precast link slab using high-performance fiber-reinforced cementitious composite (HPFRCC) reinforced with glass fiber reinforced polymer (GFRP). The suggested link slab thickness was a thin than deck slab. The study used a full-scale empirical test of three samples of full-scale expansion joints that involved part of the adjacent steel girders, as presented in Figure. 7. The beams in the experiment were loaded statically for a series of loads to simulate the traffic load and assure the elasticity response. The first link slab tested was a cast-in-place. Although this specimen displayed better continuity at the joint connecting the link slab and the deck matched to the other two specimens, it exposed to decreasing in compressive strength capacity

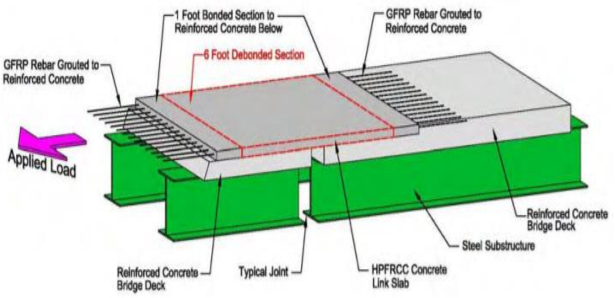

Figure.7: HPFRCC precast link slab tested by [36]

The other two link slabs were pre-cracked precast linked to the deck slab by anchors/dowels. The designers debated that the precast link slabs were beneficial that's because of the possibility of pre-cracking which led to high ductility fo link slab and enhancing the performance of the slab in compression. The disadvantage of the precasted link slab that demanded more study between link slab and bridge deck due to weak continuity. The conclusion of this research was that the precast link slab was an applicable fast solution for the bridges retrofitting. [37] Developed the link slab design by utilizing ECC and GFRP reinforcement. The featured for new link slab design a comparatively thin, the link slab element was prefabricated, consisting of (ECC) highly ductile material and reinforced by (GFRP) with low stiffness. The design details experimented by [37] is demonstrated in Figure. 8.

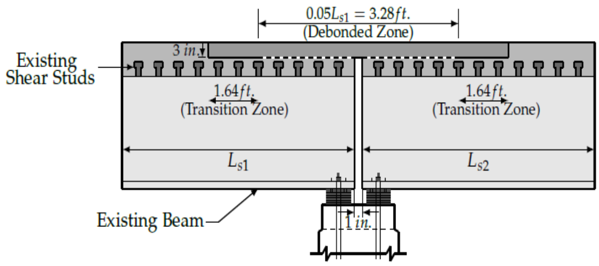

Figure.8. GFRP link slab detail

An experimental approach was executed on four-link slabs to evaluate the suggested design. The experimental research concentrated on the load-deformation response and development of cracks in flexible link elements through both monotonic and cyclic action. The specimens composed fo a two-span bridge performed a link slab installed on steel girders. They used the prefabricated and cast in place link slab. The designers concluded that the design theory showed assuring results based on load response and crack generated. In addition, the use of GFRP enhanced corrosion resistance because of its noncorrosive nature. Though the prefabricated link slab studied in this research was designed to allow both unrestrained rotations and also axial length changes, the empirical program focused only on axial deformations.

III. CONCLUSION

1- Link slabs are typically utilized for substitute deck expansion joints above piers in short and medium multi-span bridges.
2- Debonding length of up to 5 percent of the span length. The purpose of debonding is to reduce the stiffness of the link slab so that the stress developed in the link slab can be minimized
3- The continuity moment in the link slab is reduced with the increase in the ratio of debonding length. Similarly at increased bearing stiffness.
4- Taking into account the discontinuity of the lower reinforcing steel layer over the piers.
5- The link slab is subjected to combined flexural and axial loads under specific support configurations. In these cases a moment interaction diagram should be used for link slab design.
6- Link slab moments decrease with increasing beam depth. This is due to reduction in relative link slab-girder stiffness.
7- The researchers mentioned the minimum value of required reinforcement rely on AASHTO specifications to be adequate for skew link slabs at special support condition (i.e., HRRR or RRHR). Suggested additional reinforcement bottom layer to resist the high tensile stresses produced near ends of the debonded region.
Prefabricated, flexible link slab element, composed of highly ductile ECC reinforced with low stiffness GFRP, was recommended for the ideal link slab design.

REFERENCES

[1] Burke, M. P. (1991). “Flawed Assumptions: Why Bridge Deck Joints Fail." *Civil Engineering* (November), 60-62.

[2] Guthrie, W. S. (2011). "Concrete Bridge Deck Joints: State of the Practice."

[3] Alampalli, S., and Yannotti, A.P., “In-service Performance of Integral Bridges and Jointless Decks,” Transportation Research Board, No. 1624, 1998, pp. 1-7

[4] Sevgili, G. and Caner, A. (2009). Improved Seismic Response of Multisimple-Span Skewed Bridges Retrofitted with Link Slabs. Journal of Bridge Engineering, 14(6):452–459.

[5] Caner, A. and Zia, P. (1998). Behavior and Design of Link Slabs for Jointless Bridge Decks. PCI Journal, 43(3):68–80.

[6] Caner, A., Dogan, E., and Zia, P. (2002). “Seismic Performance of Multisimple-Span Bridges Retrofitted with Link Slabs.” Journal of Bridge Engineering, 7(2):85–93.

[7] Loveall, C. L. (1985). “Jointless Bridge Decks.” Civil Engineering, ASCE. 55(11), 64 -67.

[8] Philip Pierce., (1991). “Jointless Redecking.” Civil Engineering, 61(9), 6 – 61.

[9] Russell, H. G., and Gerken, L. J., (1994). “Jointless Bridges – the Knowns and the Unknowns.” Concrete International. 16(4), 44 – 48.

[10] Thippeswamy, H. K., and GangaRao, H. V. S., (1995) “Analysis of In-service Jointless Bridges.” Transportation Research Record. 1476, Transportation Research Board, Washington, D.C., 162 – 170.

[11] Thippeswamy, H. K., GangaRao, H. V. S., and Franco, J. M. (2002) “Performance Evaluation of Jointless Bridges.” Journal of Bridges Engineering, 7(5), 276 – 289.

[12] Wasserman, E, P., (2005) “Simplified Continuity Details for Short- and Medium-Span Composite Steel Girder Bridges.” Transportation Research Record. CD 11-S, Transportation Research Board, Washington, D.C., 197 – 202.

[13] Gastal, F. (1986). “Instantaneous and Time-Dependent Response and Strength of Jointless Bridge Beams.” PhD thesis, North Carolina State University, Raleigh, NC.

[14] Richardson, D. R, (1989). “Simplified Design Procedures for the Removal of Expansion Joints from Bridges Using Partially Debonded, Continuous Decks”, Master’s Thesis, North Carolina State University.

[15] El-Safty, A. K. (1994). “Analysis of jointless bridge decks with partially debonded simple span beams.” PhD Dessertation, North Carolina State Univ., Raleigh, NC.

[16] Gastal, F. (1986). “Instantaneous and Time-Dependent Response and Strength of Jointless Bridge Beams. “PhD thesis, North Carolina State University, Raleigh, NC.

[17] Caner, A., Dogan, E., and Zia, P., (2002). “Seismic Performance of Multisimple-span Bridges Retroffitted with Link Slabs.” Journal of Bridges Engineering, 7(2), 85 –93.

[18] Kowalsky, M. J., & Wing, K. M. (2003). “Analysis of an Instrumented Jointless Bridge. “(No. FHWA/NC/2002-22). North Carolina Department of Transportation.

[19] Okeil, A. M., & ElSafty, A. (2005). Partial continuity in bridge girders with jointless decks. Practice Periodical on Structural Design and Construction, 10(4), 229-238.

[20] El-Safty, A. and Okeil, A. (2008). “Extending the Service Life of Bridges using Continuous Decks.” PCI Journal, 53(6):96–111 strain-hardening cementitious composites.” (No. Research Report RC-1438).

[21] Li, V. C., Fischer, G., Kim, Y., Lepech, M. D., Qian, S., Weimann, M., & Wang, S. (2003).” Durable link slabs for jointless bridge decks based on

[22] Lepech, M. D., and Li, V. C. (2009). “Application of ECC for bridge deck link slabs. Materials and Structures.” 42(9), 1185.

[23] Li, V. C., and Fischer, G. (2002). “Reinforced ECC-An evolution from materials to structures.”

[24] Fischer, G., and Li, V. C. (2003). “Deformation behavior of fiber-reinforced polymer reinforced engineered cementitious composite (ECC) flexural members under reversed cyclic loading conditions.” Structural Journal, 100(1), 25-35.

[25] Kanda, T., Lin, Z., and Li, V. C. (1998). “Application of pseudo strain-hardening cementitious composites to shear resistant structural elements.”

[26] Sahmaran, M. Lachemi, M. Hossain K. M.A. Ranade, R. and Li, V.C. (2010). “Internal Curing of ECC’s for Prevention of Early Age Autogenous Shrinkage Cracking.” Cement and Concrete Research, 39(10), pp. 893-901

[27] Zhang, J., and Li, V. C. (2002). “Monotonic and fatigue performance in bending of fiber-reinforced engineered cementitious composite in overlay system.” Cement and Concrete Research, 32(3), 415-423.

[28] Li, V. C., Fischer, G., Kim, Y., Lepech, M. D., Qian, S., Weimann, M., & Wang, S. (2003). “Durable link slabs for jointless bridge decks based on strain-hardening cementitious composites” (No. Research Report RC-1438).

[29] Li, V. C., Lepech, M. D., and Li, M. (2005). “Field demonstration of durable link slabs for jointless bridge decks based on strain-hardening cementitious composites” (No. Research Report RC-1471).

[30] Lepech, M. D., and Li, V. C. (2008). “Large-scale processing of engineered cementitious composites”. ACI Materials Journal, 105(4), 358-366.

[31] Lepech, M. D., and Li, V. C. (2006). “Long Term Durability Performance of Engineered Cementitious Composites Restoration of Buildings and Monuments.”, 12(2), 119-132.

[32] Mothe, R. N. (2006). Partial Continuity in Prestressed Concrete Girder Bridges with Jointless Decks.

[33] Aktan, H., Attanayake, U., and Ulku, E. (2008). “Combining link slab, deck sliding over backwall, and revising bearings.” Report. No. RC-1514, Michigan Department of Transportation, Kalamazoo, MI.

[34] Ulku,E., Attanayake,U.,& Aktan, H. (2009).”Jointless Bridge Deck with Link Slabs.” Journal of Transportation Research Board, vol. 2131, pp. 68-78, 2009.

[35] Aktan, H. and Attanayake, U. (2011). “High skew link slab bridge system with deck sliding over backwall or backwall sliding over abutments.” Report. No. RC-1563, Michigan Department of Transportation, Lansing, MI.

[36] Reyes, J. and Robertson, I. N. (2011). “Precast link slab for jointless bridges decks.” Final Report. No. UHM-CEE-11-09, Hawaii Department of Transportation, Honolulu, HI.

[37] Larusson, L., Reyes, J., Lum, B., Fischer, G., and Robertson, I. N. (2013). “Development of flexible link slabs using ductile fiber reinforced concrete.” Final Report. No. UHM-CEE-13- 04, Hawaii Department of Transportation, Honolulu, HI.