1 **Development of a High Performance Concrete Deck for Louisiana's Movable Bridges:**

2 **Numerical Study**

3 **Hadi Baghi Ph.D.[1], Fatmir Menkulasi Ph.D., P.E., A.M.ASCE[2], Jacob Parker P.E.[3], Joaquim**

4 **A. O. Barros Ph.D.[4]**

5 **Abstract:** Louisiana has approximately 160 movable bridges, mostly in the southern part of the

6 state. This places Louisiana among the states with the highest inventory of movable bridges in the

7 nation. The typical deck systems in these movable bridges are steel grids. Records show that steel

8 grids have had maintenance issues. An alternative ultra-high/high performance concrete

9 (UHPC/HPC) bridge deck system is proposed for Louisiana's movable bridges. This system

10 consists of precast waffle slab deck panels, which are reinforced with glass fiber reinforced

11 polymer (GFRP) bars as positive moment reinforcement, and a two-way carbon fiber reinforced

12 polymer (CFRP) mesh as top reinforcement. Several validated nonlinear finite element analyses

13 were performed to simulate the behavior of the precast panels from the onset of loading to failure.

14 It is concluded that the precast concrete waffle slabs provide a viable alternative to steel grids by

15 supplying load capacities that surpass service level and ultimate level load demands and deflection

16 capacities that are within code specified limits.

17 [1]Post-doctoral Researcher, Department of Civil Engineering, Louisiana Tech University, Ruston, LA, 71272.
18 Email: hadibaghi@gmail.com (corresponding author)
19
20 [2]Assistant Professor, Department of Civil and Environmental Engineering, Wayne State University, Detroit, MI,
21 48202. Email: fatmir.menkulasi@wayne.edu
22
23 [3]Structural Engineer, Stantec, Baton Rouge, LA. Email: Jacob.Parker@stantec.com
24
25 [4]Professor, ISISE, Department of Civil Engineering, University of Minho, Guimarães, Portugal.
26 Email: barros@civil.uminho.pt
27
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Introduction

Louisiana has approximately 160 movable bridges, mostly in the southern part of the state. This places Louisiana among the states with the highest inventory of movable bridges in the nation. These transportation arteries are important for the economic well-being of the state, as well as for the safety of the inhabitants in hurricane vulnerable regions during evacuations. Most of the movable bridges in Louisiana are either swing-span or lift-bridge type structures. Very few movable steel bridges are of the bascule type.

The typical deck systems in movable bridges are open steel grids, which typically consist of either diagonal or rectangular grids (Fig. 1). The diagonal grids were first used in the 1920s and represent the oldest lightweight deck system (Gase 2008). The traditional steel grid decks are supported by steel stringers at typically 1.22 m on center. On average these decks weigh less than 1.20 kN/m^2; while some others can weigh as little as 0.67 kN/m^2 (Mirmiran et al. 2009). This deck system is attractive because it is light weight, the panels are prefabricated and they are easy to install and replace. Also, deck crowning, scuppers, and drains are not required, since rain water drains through the openings in the deck (Mirmiran et al. 2009). Additionally, the light weight helps with imposing as little of a demand as possible on the mechanical system. However, records show that steel grids have exhibited durability issues. The proximity of these exposed steel systems to humid environments leads to rapid deterioration. As a result, decks become loose, causing extreme noise. Furthermore, inhabitants in areas close to movable bridges often complain about noise levels resulting from vehicles crossing over the steel grids. These problems are aggravated by trapping foreign debris throughout the deck grids. The lack of traction in steel grid decks is another concern with respect to safety.

53   The Louisiana Department of Transportation and Development (LADOTD) has an interest in

54 using concrete decks to replace deteriorated steel grids on existing movable bridges as well as in

55 new construction. However, the mechanical systems of moveable bridges are highly sensitive. As

56 a result, any decking used to replace or rehabilitate the existing steel grid decking should match

57 the weight of the existing steel grid such that the mechanical system operates as designed. The

58 weight to strength ratio of conventional concrete decks will have a negative impact on the load

59 demand imposed on the mechanical system. Accordingly, a light ultra-high/high performance

60 concrete (UHPC/HPC) deck is proposed as an alternative to steel grid decking. The current

61 definition of UHPC under review in the technical committee of the American Concrete Institute

62 (ACI 239) (2015) is: "Concrete that has a minimum specified compressive strength of 150 MPa

63 with specified durability, tensile ductility and toughness requirements; fibers are generally

64 included to achieve specified requirements". High-performance concrete is defined as concrete

65 meeting special combinations of performance and uniformity requirements that cannot always be

66 achieved routinely using conventional constituents and normal mixing, placing, and curing

67 practices (ACI CT 2013). The UHPC/HPC deck system is intended to provide a continuous driving

68 surface that mimics monolithic construction, provides integral connections with the supporting

69 stringers as well as between adjacent deck panels, and provides traction, which should improve

70 traffic safety.

71   Florida Department of Transportation (FDOT) in collaboration with URS Corporation

72 identified several potential alternative lightweight solid deck systems to replace steel open grid

73 decks on typical Florida bascule bridges (Mirmiran et al. 2009; 2012; Phillips 2014; and Mirmiran

74 and Ghasemi 2016). These deck systems include a sandwich plate system, a fiber reinforced

75 polymer (FRP) composite deck, an aluminum orthotropic deck, a prismatic concrete deck with

76   FRP tubes, a non-prismatic concrete deck with FRP tubes, a FRP deck, and a waffle slab UHPC

77   deck. The sandwich plate system and the FRP composite systems are vulnerable to delamination,

78   debonding and cracking of wearing surface. The aluminum orthotropic deck is a patented product,

79   and requires expansion joints, periodic replacement of the wearing surface, galvanic corrosion

80   mitigation, as well as the potential use of blind-type fasteners (Mirmiran et al. 2009). The UHPC

81   deck with FRP tubes was not investigated with respect to the performance of panel to stringer

82   connections, panel to panel connections, and fatigue performance of the system. The deflection of

83   the FRP deck system under service load significantly exceeded the deflection limit suggested by

84   the American Association of State and Transportation Officials (AASHTO) Load and Resistance

85   Factor Design (LFRD) Specifications (2014) (Article 9.5.2). The stiffness of the FRP deck was

86   enhanced by adding an UHPC layer on the top. The dominant mode of failure in the FRP deck

87   with an UHPC overlay was either at the interface of FRP and UHPC, or through buckling of the

88   FRP web.

89   The UHPC waffle slab system was investigated further by Mirmiran et al. (2012) and Mirmiran

90   and Ghasemi (2016). Both $MMFX_2$ bars and carbon fiber reinforced polymer (CFRP) bars were

91   investigated as reinforcement alternatives for the waffle slab configuration. The overall thickness

92   of the waffle slab configuration varied from 102 mm to 127 mm, and the spacing between the

93   stringers was 1.22 m. The weight limitation was 1.01 kN/m^2. The waffle slab configuration proved

94   to be a viable alternative to steel grid decks for Florida's movable bridges by meeting AASHTO's

95   (2014) load demands.

96   As it will be described in the next section there are some differences between the characteristics

97   of the steel grid decks in Florida compared to those in Louisiana. The predominant depth of the

98   existing steel grids in Louisiana is 132 mm compared to the 102-127 mm range investigated in

4

99   Florida. The maximum weight limitation for Louisiana's movable bridge decks is 0.96 kN/m^2

100   compared to the 1.01 kN/m^2 weight limit for Florida's bascule bridge decks. The majority of

101   movable bridges in Louisiana featured steel grid spans, defined as the distance between the

102   centerlines of stringers, less than or equal to 1.27 m compared to 1.22 m in Florida. At first glance

103   these differences may appear small or negligible taking into the consideration the accuracy

104   typically employed in structural engineering work. However, given the strict limitation on the

105   maximum weight imposed on the mechanical system, such small differences required the

106   development of a unique deck configuration for Louisiana's movable bridges. The UHPC deck

107   configuration developed in Florida, while viable for Florida's bascule bridges could not be directly

108   applied to Louisiana's movable bridges for the following reasons: 1) the maximum weight limit is

109   greater than that determined for Louisiana's bridges, 2) the depth of the deck was shallower than

110   the predominant depth recorded for Louisiana's bridges, thereby increasing the overall depth

111   would further exceed the maximum weight limit, 3) the span between the centerlines of stringers

112   is 51 mm shorter than that determined for Louisiana's bridges.

113   Aaleti et al. (2011; 2013) developed an UHPC waffle slab for non-movable bridges. There was

114   no limitation on the maximum weight of the deck. The overall depth of the slab was 203 mm. The

115   thickness of the flange was 64 mm and the thickness of the transverse ribs varied from 76 mm at

116   the bottom to 102 mm at the top. The range of spans considered varied from 1.22 m to 3.05 m. The

117   weight of this deck system far exceeds the maximum weight limitation for Louisiana's movable

118   bridges (0.96 kN/m^2) because the deck configuration was developed for non-movable bridges.

119   However, the female-to-female type panel to panel connections provided a good starting point and

120   are similar to the detail proposed for Louisiana movable bridges.

121   The goal of this research is to develop an UHPC/HPC deck system for Louisiana's movable

122   bridges. A modified version of the waffle slab UHPC deck investigated in Florida is proposed for

123   investigation in this study because: 1) it can meet the limitations on weight and overall depth for

124   Louisiana's movable bridges, 2) it can meet load and deflections demands specified in AASHTO

125   (2014), 3) it uses high strength corrosion resistant reinforcement, 4) it uses panel to panel and

126   panel to stringer connections that are intended to emulate monolithic action, and 5) it includes

127   either a chip seal surface coating or a controlled broom finish to provide skid resistance and

128   increase traction.

129   **Compilation of Louisiana's current moveable bridge deck system details**

130   A list of movable bridges that utilize steel grid decking was obtained from LADOTD. The

131   bridge plans including as-built drawings and shop drawings were searched to collect all relevant

132   information such as: panel thickness, panel weight, panel length, cantilever length, span length,

133   and stringer size. Cantilever length was defined as the distance from the centerline of the exterior

134   stringer to the edge of the steel grid. Span length was defined as the distance between the

135   centerlines of stringers. The bridge plans for a total of 17 bridges were investigated during the

136   period of time allocated to complete this task. The minimum and maximum deck thickness were

137   129 mm and 150 mm, respectively. The predominant deck thickness was 132 mm. There was a

138   minimum deck weight requirement only for two bridges (0.77 kN/m^2 and 0.84 kN/m^2). The

139   maximum deck weight limitation based on stringer reactions was typically 0.96 kN/m^2 with the

140   exception of one bridge for which this limitation was 0.86 kN/m^2. The most common stringer type

141   was W410×54. The spacing of stringers varied from 1.12 m to 1.42 m, however the majority of

142   bridges featured stringer spacing less than or equal to 1.27 m.

143   Based on the collected information the following recommendations were made with the respect

144   to the development of UHPC deck panels (Table 1). The panel thickness is recommended to be

145   132 mm to be consistent with the predominant existing grid deck thicknesses. The maximum panel

146   weight should be limited to 0.96 kN/m^2 so that specified maximum stringer reactions are not

147   exceeded. The span length should be 1.27 m to cover the majority of the investigated bridges. The

148   panel length should be such that it covers at least three spans to take advantage of continuity and

149   reduce the number of joints exposed to traffic. Alternatively, continuity for superimposed loads in

150   the transverse direction can be achieved by placing a site cast UHPC/HPC diaphragm between the

151   ends of the individual precast waffle slab deck panels. This detail is illustrated in the next section.

152   The stringer size that should be considered during the development of the UHPC deck is W410×54

153   because this was the most common size.

154   **Proposed Precast Deck System**

155   The proposed system is illustrated in Fig. 2, 3 and 4. Two deck configurations are investigated

156   with respect to the number of longitudinal ribs. These two deck configurations (denoted as Deck

157   System 1 and Deck System 2 in Fig.2a) were determined based on preliminary work performed by

158   Menkulasi et al. (2016). In the first configuration there is only one partial depth longitudinal rib at

159   mid-span of the panel, which helps distribute loads to the adjacent ribs. The dimensions of this

160   partial depth rib are 22 mm wide and 70 mm deep from the bottom of the top flange. In the second

161   configuration there are a total of six partial depth longitudinal ribs. The width of these ribs is 22

162   mm whereas the depth is 19 mm. Both panel configurations will receive a chip seal surface coating

163   or a controlled broom finish to provide traction. The weight of the chip seal surface coating is

164   negligible.

165   Continuity for live loads in the transverse direction is achieved by placing a cast-in-place (CIP)

166   UHPC/HPC diaphragm between the ends of the precast deck panels when they are fabricated as

167   single span panels (Fig. 2a and Details C1a and C2a in Fig. 3 and Fig. 4). There is a distance of 24

168   mm between the end of the panels and the centerline of the stringers to allow for the placement of

169   the cast-in-place UHPC/HPC diaphragm. The top flange at the ends each panel is coped 76 mm in

170   length and 11 mm in depth to allow the C-grid to project past the ends of the panels, be immediately

171   above the coped flange and lap with the C-grid from the adjacent panels. The C-grid projects 76

172   mm past the ends of the panel to create a 152 mm lap with the grid coming for the adjacent panel

173   (Details C1a and C2a in Fig. 3 and Fig.4). All the interfaces between precast and cast-in-place

174   UHPC/HPC in the proposed connections should be sandblasted and kept moist at surface saturated

175   dry conditions to enhance bond.

176   When deck panels are fabricated such that they cover two or more spans the details will be

177   similar with the exception that the flange and transverse ribs will be continuous over the supports

178   and the cast-in-place concrete diaphragm will be placed through access holes from the top of the

179   precast deck (Fig. 2b and Details C1b and C2b in Fig. 3 and Fig. 4).

180   Continuity in the direction of traffic is provided by using female-to-female type panel to panel

181   connections and a cast-in-place UHPC/HPC fill (Details B1 and B2 in Fig. 3 and Fig.4). The steel

182   stringers are spaced at 1.27 m on center. The width of a single precast panel is 1.22 m. The overall

183   depth of the deck panels is 132 mm. The thickness of the flange is 22 mm. The width of the

184   transverse ribs, which will act as T-beams to support the superimposed loads varies; it is 51 mm

185   at the bottom of the stem and it tapers down to 22 mm (Details A1 and A2 in Fig. 3 and Fig. 4).

186   The width of end ribs at the bottom is equal to 38 mm for deck configuration 1 and 32 mm for

187   deck configuration 2.

188   The spacing of the transverse ribs is 406 mm center to center for the interior ribs and 406 mm

189   from the center of the interior rib to the outside face of the exterior rib. The weight of a single

190   panel considering the cast in place UHPC/HPC diaphragm is 0.964 kN/m^2 for deck system 1 and

191   0.955 kN/m^2 for deck system 2. The weight of deck system 1 is slightly over the target weight of

192   0.96 kN/m^2 and was calculated using a measured unit weight of 2500 kg/m^3 for *Ductal*. *Ductal* is

193   a commercial UHPC/HPC formulation provided by Lafarge North America and was supplied to

194   the research team by Lafarge as part of this study. The panel weight was calculated by ignoring

195   the presence of the reinforcement bars and mesh because the unit weight of reinforcement was less

196   than that of concrete.

197   Glass fiber reinforced polymer (GFRP) bars (GFRP V-ROD HM - 60GPa Grade III) are

198   proposed as positive moment reinforcement in the transverse and longitudinal ribs. GFRP bars are

199   corrosion resistant and are produced by Pultrall Inc. Each interior transverse rib is reinforced with

200   a No. 16 GFRP bar at the bottom and each exterior rib is reinforced with a No. 13 GFRP bar. The

201   partial depth longitudinal ribs in both configurations are reinforced with a No. 10 GFRP bar to

202   help distribute wheel loads in the longitudinal direction. The clear bottom cover to positive

203   moment reinforcement is 11 mm in the interior ribs and 10 mm in the exterior ribs. The clear cover

204   in all other cases is equal to 6 mm or greater.

205   Flange reinforcement consists of a two-way non-corrosive carbon fiber grid (C-grid)

206   developed by Chomarat North America. C-grid is an epoxy-coated composite grid made with

207   cross-laid and superimposed carbon fiber. The diameter of the strands in the C-grid is 2 mm and

208   the spacing of the strands is 41 mm in the longitudinal direction and 46 mm in the transverse

209   direction (C50 –46 x 41).

210   **Finite Element Analysis**

211   *Introduction and Validation*

212   Several nonlinear finite element analyses were performed to investigate the behavior of the

213   proposed deck panels from the onset of loading to failure. The commercially available finite

214   element analysis software Abaqus was used in all numerical simulations. The finite element

215   models were validated based on laboratory tests performed in Florida for the deck system

216   developed by Mirmiran et al. (2009). The results from the finite elements models were compared

217   with Florida's test results in terms of load displacement curves, peak loads, and crack patterns at

218   failure. Four different simulations were performed: 1) a typical T-section in a single span condition

219   (1T1S), 2) a typical T-section in a two-span continuous condition (1T2S), 3) a simply supported

220   panel that featured four spaces between the ribs (4T1S), and 4) a two span continuous panel that

221   featured three spaces between the ribs (3T2S). Single symmetry and double symmetry were

222   utilized as much as possible during the numerical simulations to reduce analysis time. Fig. 5 shows

223   the finite element mesh, boundary conditions and the double symmetry used during the simulation

224   of the 1T1S specimen.

225   3D continuum elements were used in all simulations. The size of the mesh was selected

226   such that each element side did not exceed 13 mm in length and was determined based on results

227   from convergence studies to provide a balance between accuracy and computational expense. The

228   nonlinear behavior of concrete was simulated using the concrete damage plasticity approach

229   available in Abaqus developed by Lubliner et al. (1988) and Lee and Fenves (1998). The uniaxial

230   behavior of concrete in compression and tension as well as that for the $MMFX_2$ bars was based on

231   the data reported by Mirmiran et al. (2009) and is provided in Fig. 6. The uniaxial tensile stress of

232   concrete past tensile strains of 0.01 was maintained at 1 MPa to avoid convergence issues. Table

233   2 provides the parameters used in the concrete damage plasticity model. These parameters were

234   based on calibration as well as on recommendations from Abaqus documentation (2016) and Malm

235   et al. (2006). Dilation angle represents concrete's internal friction angle. Malm et al. (2006)

236   showed that low values of dilation angle correspond to brittle behavior, while higher values

237   correspond to ductile behavior. The value of dilation angle used in this study is 45º.

238   The value of eccentricity is related to the ratio between the tensile strength and compressive

239   strength of concrete and typically is taken equal to 0.1. Therefore an eccentricity value of 0.1 was

240   adopted in this study. $\sigma_{b0} / \sigma_{c0}$ is the ratio of initial equibiaxial compressive yield stress to the initial

241   uniaxial compressive yield stress. This value was assumed to be 1.77 based on the work performed

242   by Sercombe et al. (1998). *K* is the ratio of the second stress invariant on the tensile meridian to

243   that on the compressive meridian at initial yield for any given value of the pressure invariant such

244   that the maximum principal stress is negative (Abaqus 2016). This ratio varies from 0.5 to 1.0 and

245   a typical assumed value in the concrete damaged plasticity approach is 0.67. Therefore *K* was

246   taken equal to 0.67. The viscosity parameter was assumed to be zero in this study.

247   Fig. 7 compares the experimental and numerical load versus mid-span displacement curves

248   for all four specimens. The numerically obtained load versus mid-span displacement curves for

249   specimens 1T1S, 4T1S, and 3T2S compare rather well with the experimentally obtained curves,

250   featuring only minor differences during the entire loading. The experimental and numerical curves

251   for 1T1S and 4T1S are almost identical. For the 1T2S specimen the curves are different up until a

252   displacement of 13 mm. After that the curves become very similar. It should be noted that both

253   numerical and experimental models showed an increase in stiffness for the two-span continuous

254   configurations compared to the single span configurations with the exception of tested specimen

255   1T2S, which did not show this expected trend. Considering the test setup, one possible explanation

256   for the differences between experimental and numerical load displacement curves for specimen

257   1T2S could be a delayed proper sitting of either the loading plate on the specimen or the specimen

258   on the supports.

259   Table 3 shows a comparison of peak loads obtained analytically and experimentally. The

260   average of the ratios between the numerically obtained peak load and that obtained experimentally

261   is 1.00 and the coefficient of variation is 6%. In all four cases the ratios vary from 0.92 to 1.05,

262   which indicates that the numerical models predict fairly well the capacity of the panels. Such small

263   differences in terms of peak loads suggest that the numerical model can estimate the capacity of

264   the precast panels with good accuracy. Additionally, a comparison between the numerically

265   obtained principal plastic tensile strains (PE) and crack patterns observed experimentally at failure

266   was performed for all four considered specimens. In this study the principal plastic strain contours

267   are used to qualitatively illustrate the induced damage due to crack formation and propagation

268   rather than provide an accurate depiction of crack patterns such as crack width, length, spacing,

269   etc. Such an endeavor was outside the scope of this paper. The results are shown in Fig. 8. The

270   plastic principal tensile strain contours suggest that the general location of the diagonal tension

271   crack (shear crack) in the web of the 1T1S specimen was simulated fairly well in the numerical

272   model. Additionally, two of the flexural cracks below the longitudinal rib marked in black in the

273   tested specimen match those illustrated by the principal plastic strain contours at the same location

274   in the numerical model. A diagonal shear crack was also observed during the testing of specimen

275   1T2S, and this was replicated in the numerical model near the intermediate support. In both

276   specimens 1T1S and 1T2S the failure mode was one-way shear, whereas specimens 4T1S and

277   3T2S exhibited a combination of punching shear and yield line failure. Fig. 8 shows how the

278   outline of the major crack on the top of the precast panels 4T1S and 3T2S was captured in the

279   numerical simulations.

280   The similarity in terms of load displacement curves, peak loads, and crack patterns at

281   failure suggests that the numerical models can simulate fairly well the behavior of the precast deck

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282   panel under monotonic loads. This modeling protocol was used in the development of the movable

283   bridge deck panels for the state of Louisiana. The next section provides a comparison between the

284   two proposed deck configurations in terms of load versus mid-span displacements.

285

286   ***Comparison of deck configurations 1 and 2***

287   The two proposed deck configurations were loaded to failure and their performance was

288   compared in terms of the applied load versus mid-span displacement. The investigations featured

289   a single span deck panel simply supported at the steel stringers (Fig. 9). The edges of the panels

290   were assumed to be free. This approach was taken to examine the relative performance of the two

291   deck configurations under service level and ultimate level loads in a simple setup before studying

292   the benefits of continuity and redundancy.

293   *Ductal* was used as the concrete material in both deck configurations. The uniaxial

294   behavior of *Ductal* in compression was characterized by testing in compression cylinders of 51

295   mm diameter and 102 mm height, and by obtaining the full stress-strain relationship. All specimens

296   were moist cured until the day they were tested. Fig. 10 illustrates the uniaxial stress-strain curve

297   used in the nonlinear finite element simulations. The stress-strain curve in compression covers the

298   range from the onset of loading to the peak load. The descending branch in compression was

299   conservatively ignored. The measured modulus of elasticity at 28 days (E_c = 56,312 MPa), peak

300   compressive stress (f_{cm} = 145 MPa), and the corresponding strain (ε_{cu} = 0.0026) are provided in

301   Fig. 10. The measured compressive strength for *Ductal* at 28 days did not quite meet ACI's

302   definition for UHPC (145 MPa versus 150 MPa) but it was only 5 MPa short of doing so.

303   The uniaxial behavior of *Ductal* in tension was characterized by performing splitting

304   tensile strength tests using the approach recommended by Graybeal (2006), who concluded that

13

305 an adaptation of ASTM C 496 splitting tensile test showed to provide a practical means for

306 determining the tensile cracking strength of UHPC. The splitting tensile strength tests were

307 conducted on cylinders of 102 mm diameter and 203 mm depth by recording the load that caused

308 the first crack and the peak load. The peak load was always higher than the load that caused the

309 first crack. *Ductal* was assumed to exhibit an isotropic behavior up to the formation of the first

310 crack. The measured cracking stress (f_{tm} = 11.2 MPa) and peak stress (f_{tu} =18 MPa) at 28 days are

311 provided in Fig. 10 and are similar to the values reported by Graybeal (2006) for untreated

312 specimens at 28 days. The material tested by Graybeal was also the commercial product marketed

313 by Lafarge. The strain corresponding with the peak stress (ε_{tu} = 0.0065) as well as the softening

314 modulus (E_s = 500 MPa) were based on data from direct tensile tests performed by Park et al.

315 (2012), who recorded both load and displacement to obtain the stress-strain relationship in tension

316 for ultra-high performance hybrid fiber reinforced concrete. The stress-strain curve reported by

317 Park et al. (2012) that most closely matched the first cracking and peak strength measured during

318 this study was used to correlate the peak strength to the corresponding strain. This was done due

319 to the fact that only loads were obtained during the splitting tensile test method. A tensile stress of

320 1 MPa was assumed for tensile strains exceeding 0.01 to avoid convergence issues.

321 The modulus of elasticity for the C-grid was assumed to be 234,430 MPa and was based

322 on literature provided by the manufacturer. The stress-strain relationship for the C-grid was

323 assumed to be linear, and the ultimate strain (ε_{fu}) was taken equal to 0.0099.

324 The bond between reinforcement bars and concrete was assumed to be perfect. To validate

325 this assumption the computed maximum stress on the rebars computed from finite element analysis

326 was compared with the developable stress calculated using the guidelines provided in ACI 440.1

327 (2006). The developable stress was calculated using Equation (1).

328
$$f_{fe} = \frac{0.083\sqrt{f'_c}}{\alpha}\left(13.6\frac{l_e}{d_b} + \frac{C}{d_b}\frac{l_e}{d_b} + 340\right) \le f_{fu} \tag{1}$$

329 where

330 f'_c = design concrete compressive strength (145 MPa)

331 α = bar location factor (taken equal to 1.0)

332 l_e = embedment length of reinforcement bar, (635 mm for No. 13, No. 16, No. 19, and

333 No. 22, 330 mm for No. 10).

334 d_b = diameter of reinforcement bar (10 mm for No.10, 13 mm for No.13, and 16 mm for

335 No.16).

336 C = cover to the center of the bar (19 mm for No. 16, 17 mm for No.13, 11 mm for

337 No.10).

338 f_{fu} =design tensile strength of FRP, considering reductions for service environment

339 (1372 MPa for No.10, 1312 MPa for No.13, 1184 MPa for No.16).

340   Embedment length was calculated as the distance from the point of maximum stress in the

341 reinforcement to the end of the reinforcement. Table 4 provides a summary of the maximum

342 computed stress, developable stress and the ratio between the developable stress and maximum

343 computed stress for flexural reinforcement in the longitudinal and transverse directions of the

344 bridge. The ratio between the developable stress and maximum computed stress in the GFRP bars

345 is higher than 1.0 for both transverse and longitudinal bars. These results validate the assumption

346 of a perfect bond between the reinforcement bars and concrete.

347   Both deck configurations were loaded to failure using three different positions for the truck

348 wheel loads namely a_1, a_2, and a_3 (Fig. 2). Position a_1 was used to maximize positive bending by

349 locating the center of the tire print at mid-span. Position a_2 was used to maximize one-way shear

350 effects by locating the edge of the tire print at a distance d (113 mm) from the internal face of

351   stringer support. Position a_3 was intended to maximize punching shear effects, by locating the tire

352   print in deck configuration 1 in such a way that the top flange of the deck was the only component

353   providing resistance. In deck configuration 2 the components providing resistance are the top

354   flange and the longitudinal ribs.

355   Fig. 11 shows that the performance of deck configuration 2 under the various load positions

356   is better than that of deck configuration 1. In all cases the peak load obtained for deck configuration

357   2 is higher than both the service level and ultimate level loads. All numerical simulations that do

358   not exhibit a descending branch ended at the last converged load step. Service level load (95 kN)

359   was calculated as the load corresponding to one wheel for an HL-93 truck (71 kN) times the

360   dynamic load allowance (1.33). The ultimate level load (166 kN) was calculated as the service

361   level load times the live load factor of 1.75. The difference between the peak loads obtained for

362   deck configuration 1 and 2 for tire positions a_1 and a_2 varied from 53 kN to 85 kN. For tire position

363   a_3 the difference between the peak loads was 75 kN. Deck configuration 2 exhibited larger

364   deflection capacity compared to deck configuration 1. Such deflection capacity is essential when

365   the deck is subject to ultimate level loads because it provides an opportunity for the adjacent ribs

366   or deck panels to share the load once cracking, yielding or softening takes place at the most critical

367   location. In general, deck configuration 2 was stronger and more ductile but it also softened at

368   lower loads. For example for load position a_1 the stiffness of the deck configuration 2 is noticeably

369   lower than the stiffness of deck configuration 1 even for loads lower than the service level load.

370   Because deck configuration 2 performed much better than deck configuration 1, this configuration

371   was selected for all subsequent analyses.

372   The influence of tire position on the capacity of the deck panel was further examined by

373   investigating three additional loading positions namely b_1, b_2, and b_3 on deck configuration 2 (Fig.

16

374   2). These additional tire positions match with tire positions a_1, a_2, and a_3 with respect to their

375   locations from the stringer supports, however the wheel loads in these cases are centered over the

376   ribs rather than in between them. Fig. 12 suggests that the peak loads for load positions *a* and *b*

377   were generally similar. The differences in peak loads varied from 13 kN to 22 kN.

378   Fig. 13 illustrates the principal plastic tensile strains at failure for all load positions

379   described so far. The black color represents principal plastic tensile strains that are equal to or

380   greater than 0.0065, which is the strain that corresponds with the peak tensile strength measured

381   during the material characterization study. Load position a_1 featured mid-span loading with the

382   load placed between the transverse ribs. As a result, there is significant cracking in the flange and

383   in the longitudinal ribs immediately underneath the load. Once the load is transferred to the

384   transverse ribs the majority of the cracks take place in the webs of the transverse ribs due to their

385   reduced thickness. Additionally, there are some flexural cracks in the bottom of the transverse ribs

386   at mid-span. The overall behavior of the deck panel for this load position can be characterized as

387   follows: flexural cracking will initiate at the bottom of longitudinal ribs followed by flexural

388   cracking at the bottom of top flange and bottom of stem; formation of shear cracks in the stem will

389   ensue; the ultimate condition is expected to be a shear failure of the stem.

390   For load position a_2 the principal plastic tensile strain contours in the flange are similar to

391   those observed for load position a_1. However, as expected, the plastic strain contour in the webs

392   of the transverse ribs is more pronounced in the more heavily loaded side of the span. In addition

393   to the shear cracks in the web of the transverse ribs, there are also shear-flexural cracks at the

394   bottom of the transverse ribs, flexural cracks near mid-span, as well as some cracking at the bottom

395   of the transverse ribs near the support. The overall behavior of the deck panel for this load position

17

396   in terms of the sequence of cracking and the ultimate condition is similar to that described for load

397   position a_1.

398   For load position a_3 the cracking is concentrated in the top flange as well as in the webs of

399   the transverse ribs. Cracking in the transverse ribs is dominated by shear cracking in the web. The

400   failure mode for this load position is expected to be dominated by shear in the stem with the deck

401   panel exhibiting significant flexural cracking at the bottom of the top flange.

402   The principal plastic tensile strain contours for load position b_1 differ from those for load

403   position a_1 because the majority of the cracks take place in the web of the loaded transverse rib as

404   well as in the bottom of the stem as opposed to the top flange. This is due to the fact that the load

405   is centered over the ribs. As stated earlier, cracking in the webs of the transverse ribs is dominated

406   by shear cracks and takes place due the reduced thickness of the web, while cracking in the bottom

407   of the stem features flexure-shear and flexure cracks. The overall behavior of the deck panel for

408   this load position can be characterized as follows: flexural cracking will initiate at the bottom of

409   stem, followed by the formation of shear cracks in the stem; the ultimate condition is expected to

410   be a shear failure of the stem.

411   The principal plastic tensile strain contours for load position b_2 are characterized mainly

412   by shear cracks in the webs of the most heavily loaded transverse ribs, as well as some flexure-

413   shear cracks and flexure cracks at the bottom of the stem. The cracking in the top flange is not as

414   severe as it is for load position a_2 because the load is applied directly over the transverse ribs. The

415   overall behavior of the deck panel for this load position in terms of the sequence of cracking and

416   the ultimate condition is similar to that described for load position b_1.

417   Finally, the principal plastic tensile strain contours for load position b_3 suggest shear

418   cracking in the webs of the transverse ribs near the support. The overall behavior of the deck panel

419   for this load position is characterized by the formation of shear cracks in the stem, followed by

420   flexural cracking at the bottom of the stem; the ultimate condition is expected to be a shear failure

421   of the stem. The next section describes the details of a parametric study that was undertaken for

422   deck configuration 2 to investigate the influence of a variety of parameters on the performance of

423   the deck panel.

424   ***Parametric study***

425   Given that deck configuration 2 showed promise in providing an alternative solution for

426   Louisiana's movable bridge decks, the influence of a variety of parameters on the performance of

427   the deck panel was investigated. These parameters were: 1) the addition of CFRP stirrups in the

428   webs of the transverse ribs, 2) the influence of more than one layer of C-grid in the top flange, and

429   3) the influence of the uniaxial behavior of *Ductal* in tension on the behavior of the deck panel in

430   terms of strength and stiffness. The single span deck panel simply supported at stringer locations

431   illustrated in Fig. 9 was used for the parametric study to reduce analysis time.

432   Fig. 14 shows the investigated layout of CFRP shear reinforcement in the transverse ribs.

433   The CFRP shear reinforcement consists of the C-grid used as top flange reinforcement. Loading

434   position b_1 was used to investigate the influence of CFRP shear reinforcement on the behavior of

435   the deck panel. Fig. 15a shows that the influence of CFRP stirrups on the performance of the deck

436   panel is negligible. This suggests that the contribution of the CFRP shear reinforcement to the

437   strength of the deck panel compared to the contribution of concrete is much smaller. *Ductal*'s

438   contribution to the shear strength of the panel is provided in terms of compression struts and

439   tension ties. The area of CFRP shear reinforcement is not large enough to make a marked

440   difference. Accordingly, given the small thickness of the web and the fact that the placement of

441 such shear reinforcement will require additional labor and quality control the addition of this type

442 of shear reinforcement is not recommended.

443   Although all the deck panels investigated so far featured single panels simply supported at

444 the stringer locations, the influence of the C-grid on helping the top flange distribute loads between

445 the transverse ribs was investigated by varying the presence and amount of carbon fiber

446 reinforcement. Three cases were analyzed: 1) no C-grid, 2) one-layer of C-grid, and 3) two layers

447 of C-grid. Tire position a_1 was considered in all cases because it featured a load position between

448 the transverse ribs. Fig. 15b suggests that the C-grid increases slightly the capacity of the deck

449 panel. The difference in peak loads is 6 kips for cases that featured no grid and one layer of grid,

450 and 3 kips for cases that featured one layer of grid and two layers of grid. Additionally, the C-grid

451 serves as negative moment reinforcement in regions of negative moment and also helps in

452 controlling crack widths. It is therefore recommended to include at least one layer of the C-grid as

453 reinforcement in the top flange.

454   Finally, the influence of the uniaxial behavior of *Ductal* in tension on the performance of

455 the deck panel was investigated by examining three different uniaxial tensile behaviors. One of the

456 advantages of UHPC is that it can provide considerable resistance in tension compared to normal

457 strength concrete. However, the behavior of UHPC in tension varies widely and depends on fiber

458 orientation as well as the test method used to characterize this behavior. Fig. 16a illustrates the

459 three uniaxial stress-strain relationships in tension considered for this study. The first is the tensile

460 behavior of *Ductal* used in the study performed by Mirmiran et al. (2009), which is characterized

461 by linear elastic behavior up until the first crack and a slight softening behavior after the first crack.

462 The second is similar to the first except that the cracking stress is higher. The softening modulus

463 for both of these cases was assumed to be 35 MPa and the maximum tensile strain was limited to

464   0.01. The cracking stress for the second tensile behavior was based on the measured value during

465   the splitting tensile tests performed for this study. The third behavior is characterized by linear

466   elastic behavior up until the first crack, a hardening branch between the cracking stress and the

467   peak stress, and a softening branch past the peak tensile strength. As stated earlier the softening

468   modulus for third case was 500 MPa and was based on data collected by Park et al. (2012). The

469   maximum tensile strain for the third case was also limited to 0.01. For tensile strains greater than

470   0.01 a tensile stress of 1 MPa was assumed to avoid convergence issues.

471   Fig. 16b suggests that the uniaxial tensile behavior of *Ductal* affects the overall behavior

472   of the deck panel under the applied load. The higher the cracking stress the higher the capacity of

473   the deck panel. Additionally, the hardening behavior after the first crack leads to a higher capacity

474   for the deck panel. While the difference in peak loads is not significant an improved uniaxial

475   behavior in tension leads to a stiffer and slightly stronger response of the deck panel.

476   *Effect of Continuity*

477   All the numerical simulations performed so far were done on single span deck panels

478   simply supported at the stringer supports. In reality the deck for the movable bridges will consist

479   of several precast deck panels in both the longitudinal and transverse directions of the bridge. To

480   investigate the influence of continuity on the load capacity of the deck panel a two span continuous

481   single deck panel configuration was investigated (Fig. 17a). Various load positions were

482   investigated with the purpose of selecting the one that leads to the lowest peak load for the two-

483   span continuous configuration. These load positions are illustrated in Fig. 17a and feature two HL-

484   93 truck wheel loads spaced 1.83 m apart. Each load position features wheel loads centered over

485   the transverse ribs. This orientation of wheel loads was previously determined to result in the

486   lowest peak load for the single span precast panels. The first two selected load positions were c_1

487   and c_3. Load position c_1 was expected to maximize shear stresses at the exterior left support. Load

488   position c_3 was expected to maximize flexural positive stresses and deflections in the left span.

489   Load position c_2 was added to capture a loading case between load position c_1 and c_3. Load position

490   c_4 was expected to maximize shear stresses at the interior support. Load position c_5 represents a

491   case in which half of the wheel load is supported by one panel and the other half by the adjacent

492   panel. Fig. 17b suggests that load positions c_1, c_4, and c_5 do not control, and that the most critical

493   cases are load positions c_2 and c_3 (i.e. when one of the wheel loads is placed near mid-span). Load

494   position c_5 did not control because the transverse joint features two exterior reinforced ribs side by

495   side. The differences in load displacement curves between load position c_2 and c_3 are small enough

496   to suggest that additional load cases that feature tire positions between c_2 and c_3 are not warranted.

497   For each load position a nonlinear finite element analysis was performed to obtain the full load

498   versus mid-span displacement curve. These curves are illustrated in Fig. 17b. The vertical axis

499   represents the ratio between the load applied to the deck panels and the ultimate level load for each

500   case. In some cases only a portion of the two wheel loads could be applied to the two span

501   continuous configuration because the spacing of the wheel loads specified in AASHTO is 1.83 m

502   and the span length for the deck panels is 1.27 m. The load position that led to the lowest peak

503   load was load position c_3. This load position was used to investigate the efficiency of the C-grid

504   as negative moment reinforcement.

505   The influence of the C-grid as negative moment reinforcement was investigated by varying the

506   presence and amount of the carbon fiber reinforcement in the deck panel for load position c_3 (Fig.

507   18a). The service level load was calculated by multiplying the portion of the wheel loads that fell

508   on the two span continuous configuration by the dynamic load allowance (1.33). The ultimate level

509   load was calculated by multiplying the service level load by 1.75. Fig. 18b illustrates the

510   relationship between the total load versus the displacement at the center of the load in the left span.

511   When no layer of C-grid is provided, the peak load (270 kN) is slightly smaller than the ultimate

512   level load (273 kN). When one layer of C-grid is provided, the peak load is increased to 301 kN,

513   and when two layers are provided the peak load is 319 kN. Fig. 18b suggests that there needs to

514   be at least one layer of C-grid in the top flange to meet the ultimate load demand.

515   The ratio between the peak load (301 kN) obtained for load position c_3 in the two span

516   continuous configuration and the ultimate level load (273 kN) is 1.10. This is similar and actually

517   lower than the ratio between the peak load (187 kN) for load position b_1 in the single deck singly

518   supported configuration and the corresponding ultimate level load (166 kN), which is 1.13. This

519   suggests that the ultimate condition is dominated by shear failure and that continuity in the

520   transverse direction does not help in increasing the load capacity of the deck panels. The inclusion

521   of adjacent panels in the direction of traffic may or may not increase the load capacity of the deck

522   system. This was not evaluated because it was outside the scope of work and because the service

523   level and ultimate level loads were surpassed in all investigated cases. The fact that the mode of

524   failure is primarily a shear failure is illustrated in Fig. 19a, which provides an isometric view of

525   the two span continuous configuration from the bottom and depicts the principal plastic tensile

526   strain contours. Regions in black represent areas in which the principal tensile plastic strain is

527   equal to or greater than 0.0065 and are located primarily in the webs of the transverse ribs.

528   Finally, all load displacement curves shown so far demonstrate that the proposed deck panel

529   configuration possesses significant deflection capacity in the nonlinear range regardless of the load

530   position and continuity. In all investigated cases the maximum recorded deflection was at least 20

531   mm. The softening of the load displacement curve typically occurred at a mid-span deflection of

532   approximately 1 mm. As a result, the proposed deck configuration offers a deflection capacity that

533   is at least 20 times the computed elastic displacement.

534   Fig. 19b illustrates the principal plastic tensile strain contours at failure for the two span

535   continuous configuration for two cases. The first case features no C-grid reinforcement in the top

536   flange (left), and the second features one layer of C-grid in the top flange. Only the strain contours

537   for the left span in Fig. 18a are shown. The black color represents those regions in which the plastic

538   strain is higher than or equal to 0.0065, which is the strain that corresponds with the peak tensile

539   strength recorded during the material characterization study. The principal tensile plastic strain

540   contours corroborate the conclusion drawn earlier that there should be at least one layer of C-grid

541   in the top flange. The extent of damage in terms of crack formation is more pronounced in the case

542   that features no layers of C-grid in the top flange. As a result, the presence of the C-grid in the top

543   flange helps control the extent of cracking in addition to increasing the load capacity of the deck

544   panel.

545   *Deflections*

546   Fig. 20 shows the load versus mid-span displacement for the single span simply supported

547   condition and load position b_1 as well as for the two span continuous configuration and load

548   position c_3 up to and beyond service level loads. Load positions b_1 and c_3 led to maximum

549   deflections at service for the single span and two span continuous configurations, respectively. The

550   dashed vertical line represents the maximum allowable deflection for steel grid decks specified in

551   Article 9.5.2 of AASHTO LFRD Specifications (2014), which was calculated as L/800. For the

552   simply supported condition the deflection at service is 5.5 mm, which is greater than the allowable

553   limit (1.6 mm). However, when continuity is introduced the deflection at service becomes 2.2 mm,

554   which is 38% larger than the allowable limit. This suggests that while the introduction of continuity

24

555   did not result in an increase in terms of load capacity for the panel, it resulted in an increase in

556   stiffness. The No.16 bars in the interior transverse ribs were replaced with No.19 and No.22 bars

557   to study their influence on the stiffness of the deck panels. Additionally, the bars in the exterior

558   transverse ribs were changed to No. 16 for both cases.   When a No.19 bar is used, the maximum

559   deflection in the two span continuous configuration is 1.7 mm as opposed to the 1.6 mm allowable.

560   When a No.22 bar is used the maximum deflection is 1.5 mm, which is smaller than the allowable

561   limit. The perfect bond assumption was validated for both new bar sizes and the results are shown

562   in Table 4. These results suggest that simply increasing the size of the bottom bars helps satisfy

563   AASHTO's requirement for deflection. Also, it is expected that the introduction of additional

564   panels, both, in the transverse and longitudinal directions should result in deflections that are even

565   lower than the allowable limit.

566   In the light of this discussion it is recommended that the deck panels be fabricated in at least a

567   two span continuous configuration to increase the stiffness of the system by taking advantage of

568   the inherent continuity in such a configuration. Additionally, it is recommended that No. 22 bars

569   be used to meet AASHTO's deflection limit. Even in cases for which AASHTO's deflection limit

570   may not be critical to the performance of the deck system it is recommended that the deck panels

571   be fabricated in at least a two span continuous configuration to reduce the number of joints exposed

572   to traffic.

573   **Conclusions and Recommendations**

574   The development of a high performance concrete deck for movable bridges in the state of

575   Louisiana was presented. The study was analytical in nature and consisted of several validated

576   nonlinear finite element analyses. The primary challenge in developing this deck system was the

577   limitation on the overall weight of the deck panel.

578   Two deck configurations were investigated. The first deck configuration failed to meet ultimate

579   load demands for those load positions that maximized the effects of one-way shear and punching

580   shear. The second deck configuration performed satisfactorily under all load positions by

581   providing load capacities that surpassed service level and ultimate level loads. This deck

582   configuration was used in all subsequent analyses.

583   *Ductal* produced by Lafarge North America was used as the concrete material. An improved

584   uniaxial behavior of *Ductal* in tension appears to improve the overall behavior of the deck panel

585   under load. GFRP bars are recommended as reinforcement for the bottom of the stem because they

586   are light, corrosion resistant, can be fully developed in the available space, and are able to help

587   develop the computed deck panel capacities. One layer of C-grid is recommended to be used as

588   top flange reinforcement to provide negative moment resistance, control cracking, and distribute

589   loads in the longitudinal direction.

590   The presence of the C-grid as shear reinforcement had a negligible effect on the capacity of

591   the deck panel. Accordingly, while the proposed deck configuration satisfies AASHTO's ultimate

592   load demand without any shear reinforcement, other potential shear reinforcement options will be

593   considered during future research provided that shear was critical for the UHPC/HPC waffle slabs.

594   Continuity in the transverse direction did not result in an increase in deck panel capacity

595   compared to the corresponding ultimate load, but it increased the stiffness of the deck system. This

596   increase in stiffness resulted in lower deflections at service. It is recommended that the deck panels

597   be fabricated in at least a two span continuous configuration to increase the stiffness of the system

598   by taking advantage of the inherent continuity is such a configuration. Additionally, it is

599   recommended that No. 22 bars be used for the interior transverse ribs to meet AASHTO's

600   deflection limit (Article 9.5.2). Even in cases for which AASHTO's deflection limit may not be

601 critical to the performance of the deck system it is recommended that the deck panels be fabricated

602 in at least a two span continuous configuration to reduce the number of joints exposed to traffic.

603 The ultimate condition determined based on the principal plastic tensile strain contours at peak

604 loads was dominated by shear failure in the webs of the transverse ribs, with the deck panel

605 exhibiting flexural cracking at the bottom of the stem and bottom of the top flange. Flexural

606 cracking in the top flange was more pronounced for those load positions which featured wheel

607 loading between the transverse ribs.

608 The analytical investigations presented in this paper were based on several assumptions.

609 Physical testing of single span and multiple span deck panels is scheduled in the near future to

610 validate some of the assumptions made during this analytical study. Additionally, in all cases the

611 loading was monotonic. Physical testing of the deck panels under cyclic loading will be conducted

612 to investigate the effects of fatigue on the performance of the deck panels. Also, additional concrete

613 mixes will be investigated and alternative deck panel configurations will be developed with the

614 purpose of arriving at an option that provides the best balance between performance and economy.

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Diagonal grid steel deck   Rectangular grid steel deck

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726 a) Single span precast deck panels

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728 b) Two-span continuous precast deck panels
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730

406 mm   406 mm   406 mm   406 mm   406 mm   406 mm

132 mm

76 mm   76 mm   76 mm

C-GRID OR FIBERGLASS MESH

SEE DETAIL A1
SEE DETAIL B1

LONG. SECTION ALONG BRIDGE-DECK SYSTEM 1
PARALLEL TO TRAFFIC (MID SPAN)

22

22   70   14   26

C-GRID

11 (typ.)

51

No. 16 mm
V-ROD

DETAIL A1

C-GRID

635 mm   635 mm   1270 mm

W410x54   DETAIL D1   DETAIL C1

ELASTOMERIC BEARING PAD

TRANS. SECTION - DECK SYSTEM 1
PERPENDICULAR TO TRAFFIC (MID SPAN)

Cast-in-place UHPC/HPC fill
TOP OF PANEL

6   25   6

152 mm lap between the C-GRIDs

17   6   13   6

132 mm   22   70   40

22   10

38   38

Precast deck panel

No. 13 mm V-ROD

DETAIL B1

152 mm lap between the C-grids
Cast-in-place UHPC/HPC diaphragm
C-GRID

24

11 mm

76   76

Precast deck panel

10 mm DIA. Headed stud

6 mmx 64 mm CONTINUOUS ELASTOMERIC BEARING PAD

105 mm

DETAIL C1a
(Single span precast deck panels)

Cast-in-place UHPC/HPC diaphragm
C-GRID

24

Precast deck panel

10 mm DIA. HEADED STUD

6 mmx 64 mm CONTINUOUS ELASTOMERIC BEARING PAD

105 mm

DETAIL C1b
(Two-span continuous precast deck panels)

No.10 mm V-Rod

6 mm

22

70

22

DETAIL D1

731

35

Figures showing bridge-deck system details including:

406 mm (×6) · C-GRID · 132 mm · 76 mm · 76 mm · 76 mm

SEE DETAIL A2 · SEE DETAIL B2 · LONG. SECTION ALONG BRIDGE-DECK SYSTEM 2 · PARALLEL TO TRAFFIC (MID SPAN)

C-GRID · 205 TYP. · 208 TYP. · 1270 mm (TYP.) · W410x54 · DETAIL D2 · DETAIL C2 · ELASTOMERIC BEARING PAD · PERPENDICULAR TO TRAFFIC (MID SPAN) · TRANS. SECTION – DECK SYSTEM 2

DETAIL A2 · 22 · 22 · 19 · 51 · 14 · 26 · 51 · C-GRID · 11 (Typ.) · No.16 mm V-ROD

TOP OF PANEL · Cast-in-place UHPC/HPC fill · 152 mm lap between the C-grids · 22 · 6 · 25 · 6 · 19 · 132 mm · 91 · 17 · 6 · 13 · 10 · 22 · Precast deck panel · 32 · 32 · No.13 mm V-ROD · DETAIL B2

152 mm lap between the C-grids · Cast-in-place UHPC/HPC diaphragm · C-GRID · 24 · 11 mm · Precast deck panel · 76 · 76 · 105 mm · 10 mm DIA. HEADED STUD · 6 mmx 64 mm CONTINUOUS ELASTOMERIC BEARING PAD · DETAIL C2a · (Single span continuous precast deck panels)

Cast-in-place UHPC/HPC diaphragm · C-GRID · 24 · Precast deck panel · 105 mm · 10 mm DIA. HEADED STUD · 6 mmx 64 mm CONTINUOUS ELASTOMERIC BEARING PAD · DETAIL C2b · (Two-span continuous precast deck panels)

No.10 mm V-Rod · 6 mm · 22 · 22 · 19 · DETAIL D2

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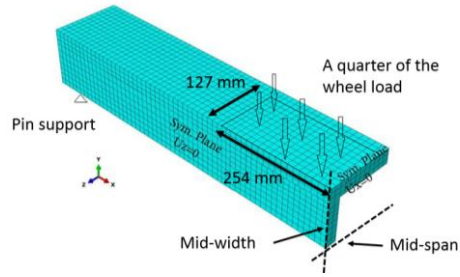
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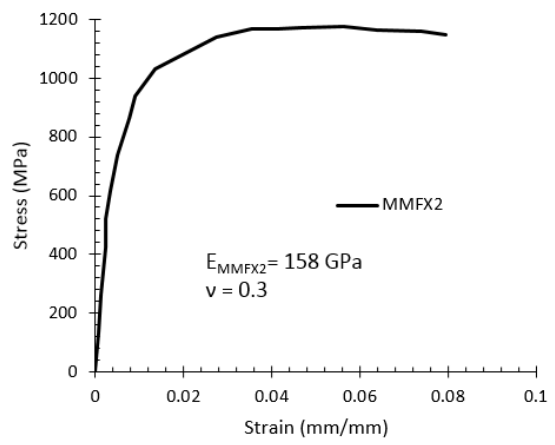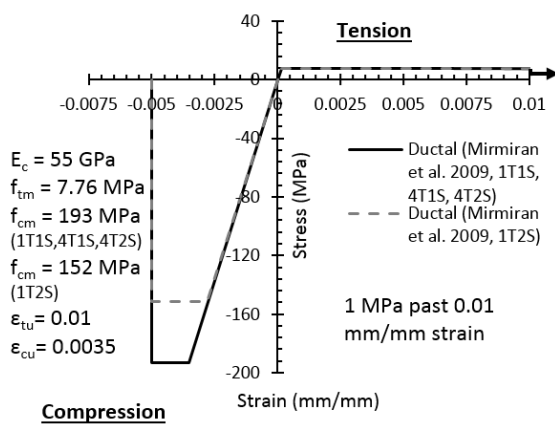
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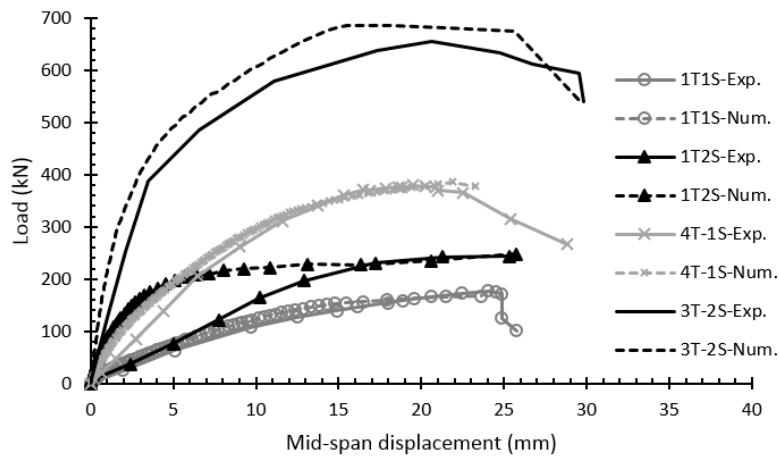
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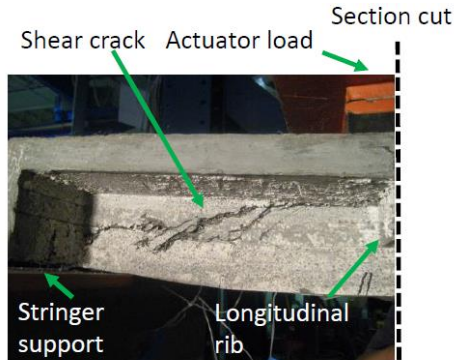
Experimental crack pattern (1T1S)
(Test performed by Mirmiran et al. (2009))
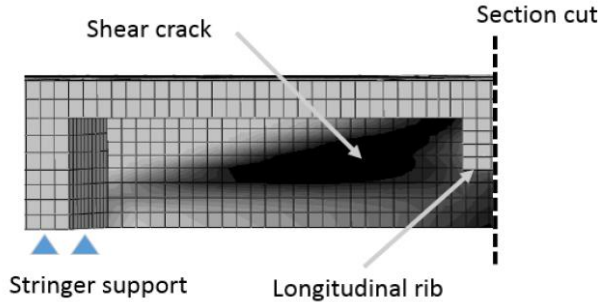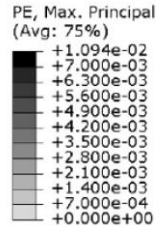
Simulated principle plastic tensile strain contours(1T1S)

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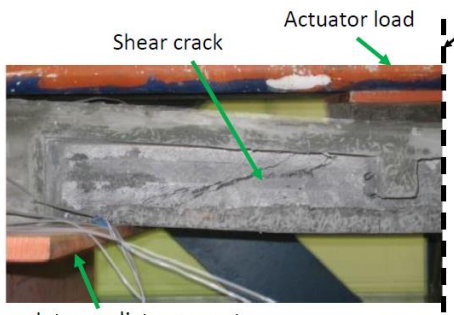
Experimental crack pattern (1T2S)
(Test performed by Mirmiran et al. (2009))
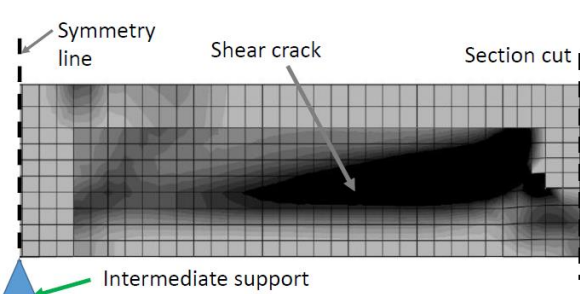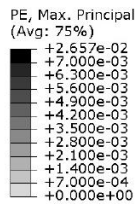
Simulated principle plastic tensile strain contours (1T2S)
(in black color: $\varepsilon_{tu} \geq 0.0065$)

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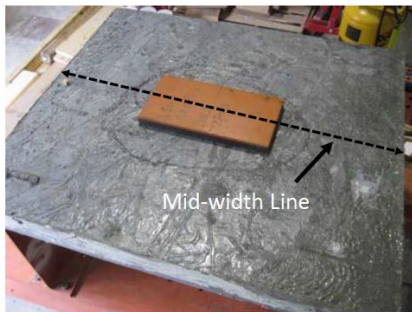
Experimental crack pattern (4T1S)
(Test performed by Mirmiran et al. (2009))
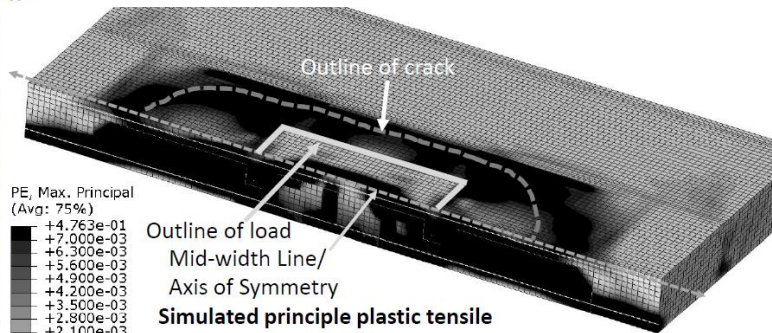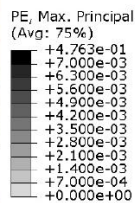
Simulated principle plastic tensile
strain contours (4T1S)
(in black color: $\varepsilon_{tu} \geq 0.0065$)

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Position of the Load

Mid-width Line/Axis of Symmetry
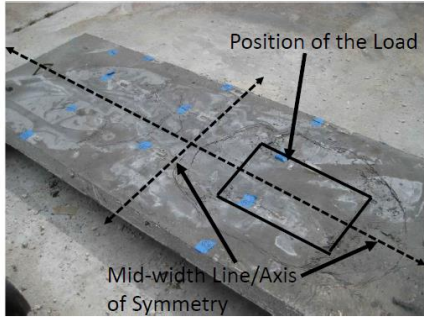
Experimental crack pattern (3T2S)
(Test performed by Mirmiran et al. (2009))

Outline of load

Outline of crack
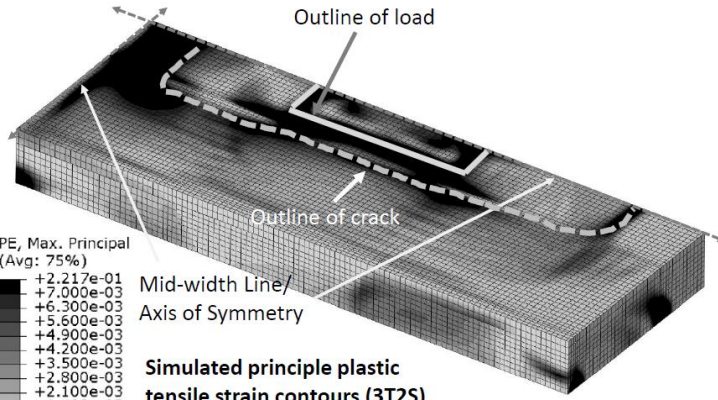
PE, Max. Principal
(Avg: 75%)
+2.217e-01
+7.000e-03
+6.300e-03
+5.600e-03
+4.900e-03
+4.200e-03
+3.500e-03
+2.800e-03
+2.100e-03
+1.400e-03
+7.000e-04
+0.000e+00

Mid-width Line/
Axis of Symmetry

Simulated principle plastic
tensile strain contours (3T2S)
(in black color: $\varepsilon_{tu} \geq 0.0065$)

Steel stringer

Free edge

Deck Panel

Pin
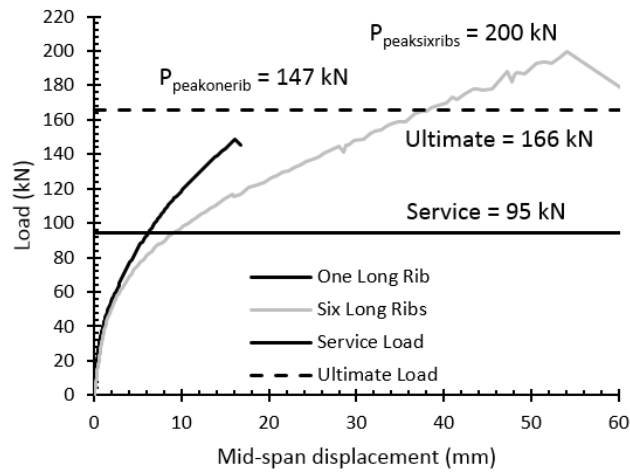
Roller

Free edge

1220 mm

1270 mm

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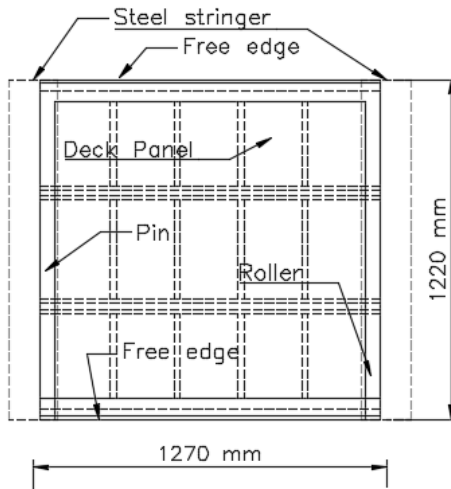
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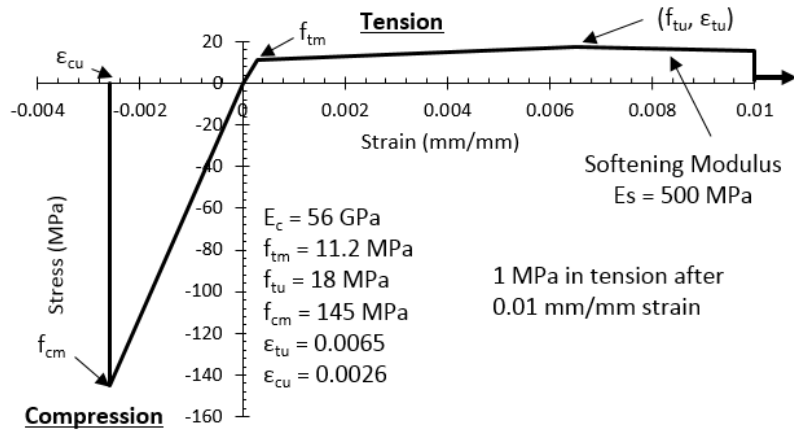
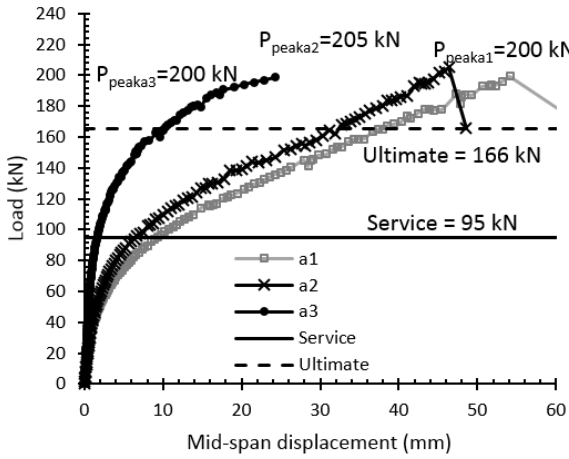844
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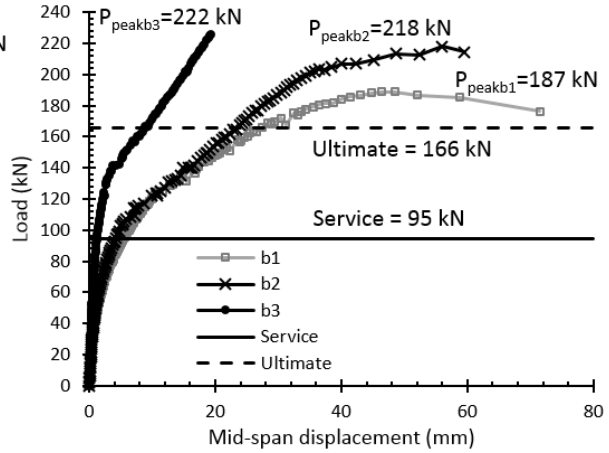853
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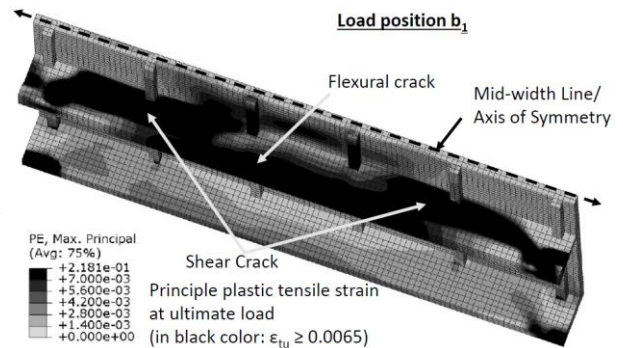

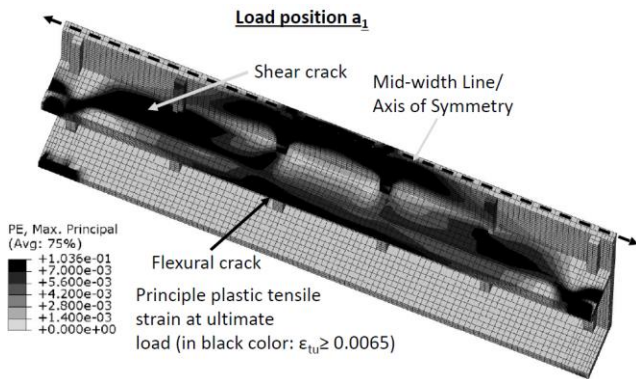   b)
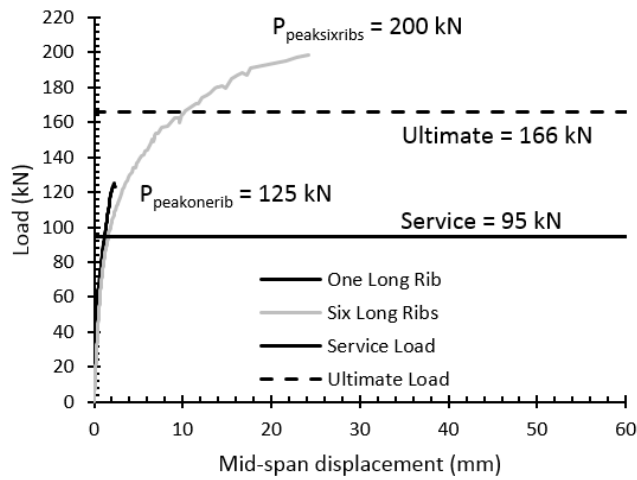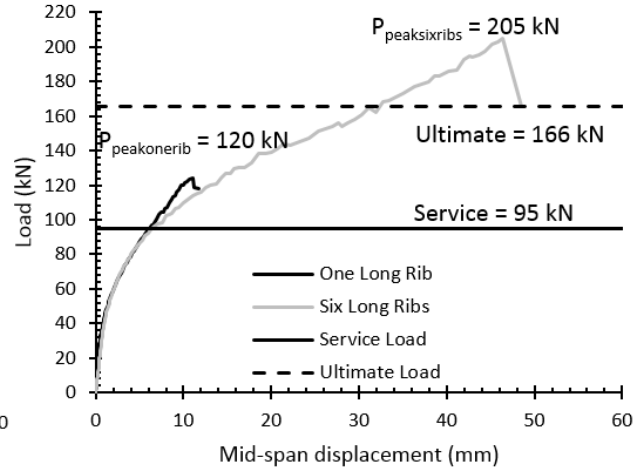
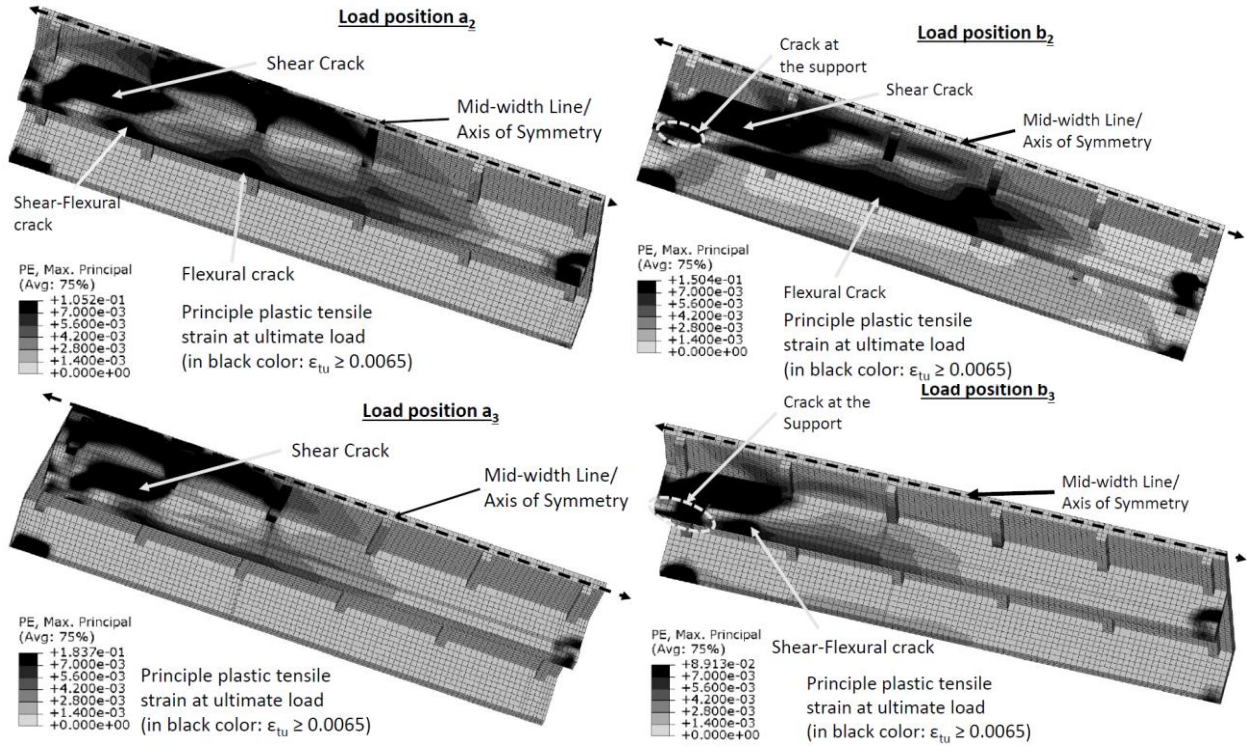
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Load position a₂

Shear Crack

Mid-width Line/
Axis of Symmetry

Shear-Flexural
crack

PE, Max. Principal
(Avg: 75%)
+1.052e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Flexural crack

Principle plastic tensile
strain at ultimate load
(in black color: $\varepsilon_{tu} \geq 0.0065$)

Load position b₂

Crack at
the support

Shear Crack

Mid-width Line/
Axis of Symmetry

PE, Max. Principal
(Avg: 75%)
+1.504e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Flexural Crack

Principle plastic tensile
strain at ultimate load
(in black color: $\varepsilon_{tu} \geq 0.0065$)

Load position a₃

Shear Crack

Mid-width Line/
Axis of Symmetry

PE, Max. Principal
(Avg: 75%)
+1.837e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Principle plastic tensile
strain at ultimate load
(in black color: $\varepsilon_{tu} \geq 0.0065$)

Load position b₃

Crack at the
Support

Mid-width Line/
Axis of Symmetry

PE, Max. Principal
(Avg: 75%)
+8.913e-02
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Shear-Flexural crack

Principle plastic tensile
strain at ultimate load
(in black color: $\varepsilon_{tu} \geq 0.0065$)

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C-Grid

Transverse rib   Longitudinal rib

Stringer (Typ.)

Elevation

Top mesh

C-Grid

Bottom bar

Cross-section
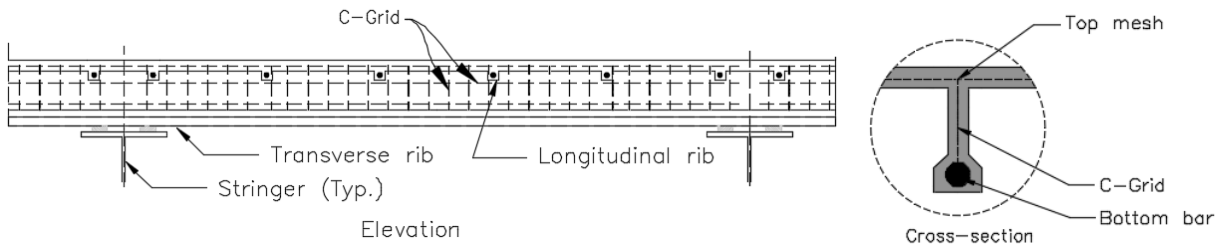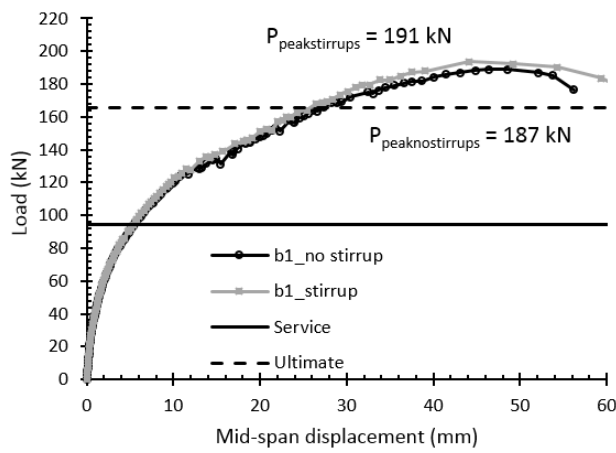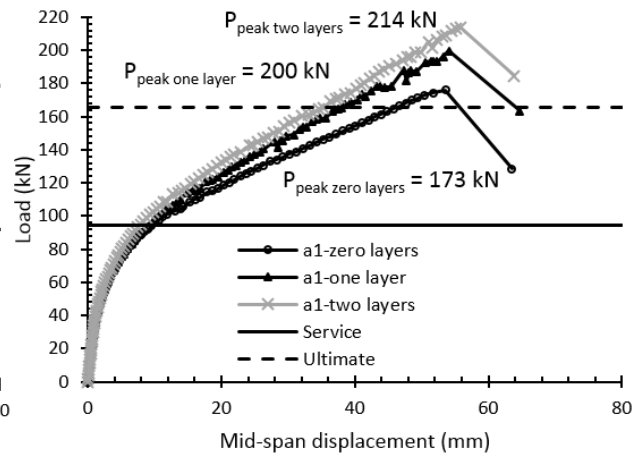
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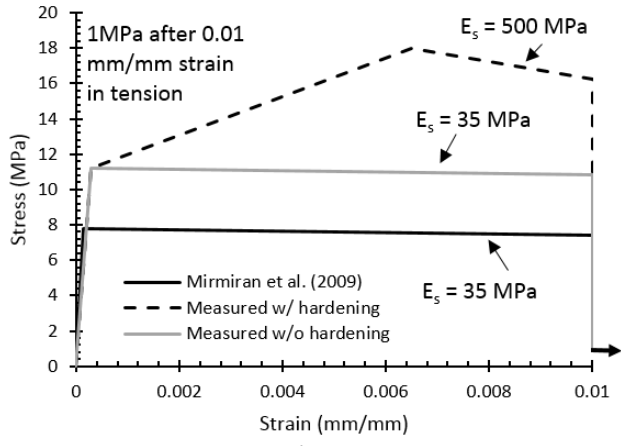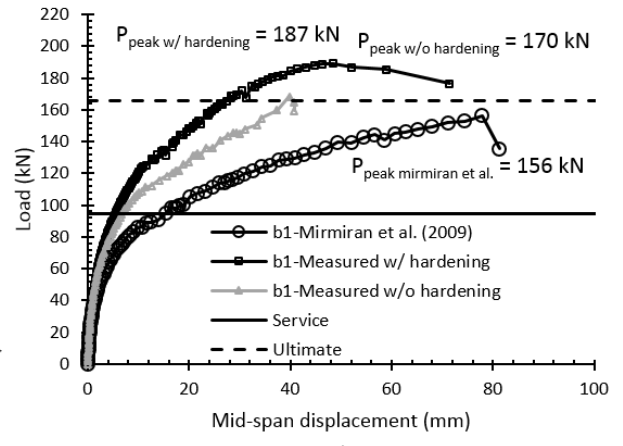
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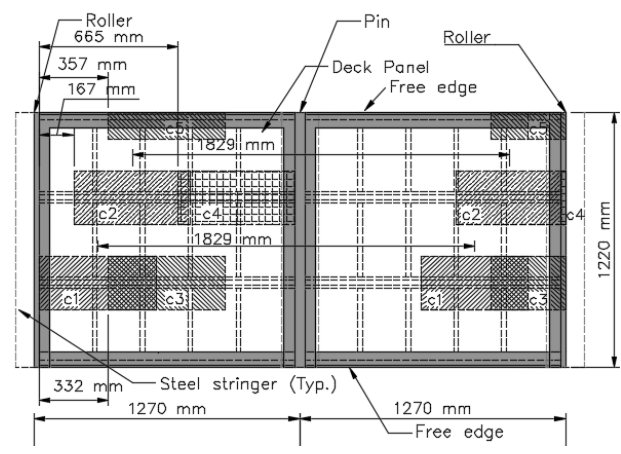941
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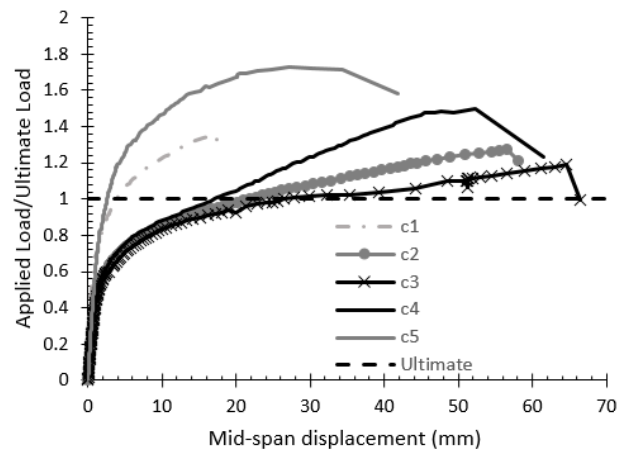957   a)   b)
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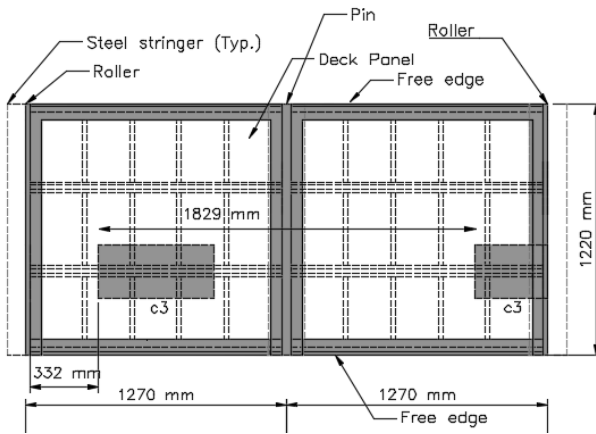964
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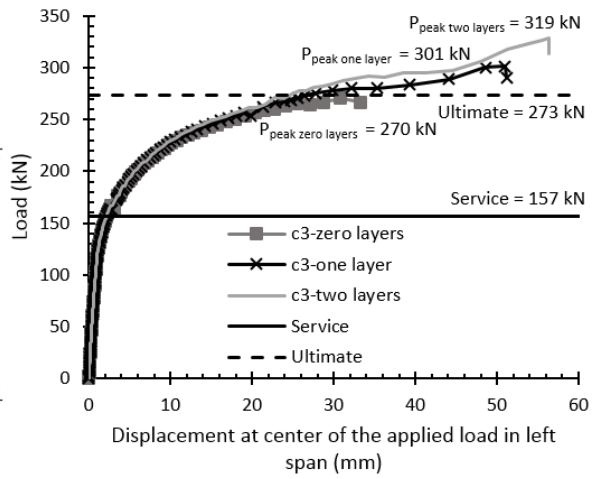983
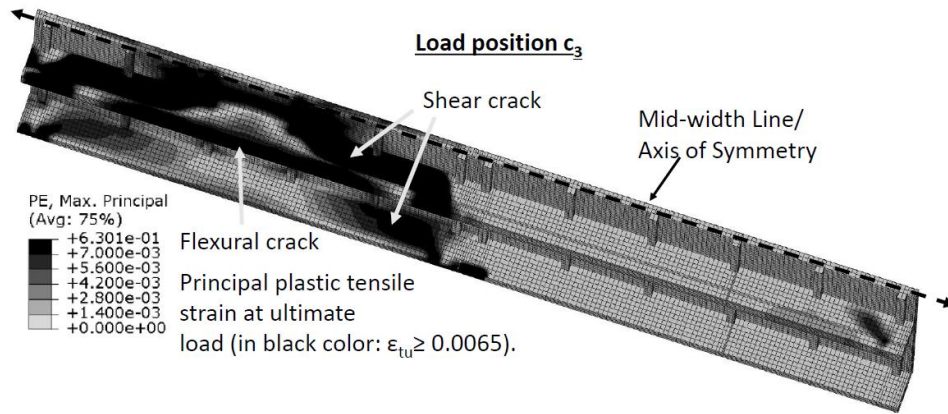984
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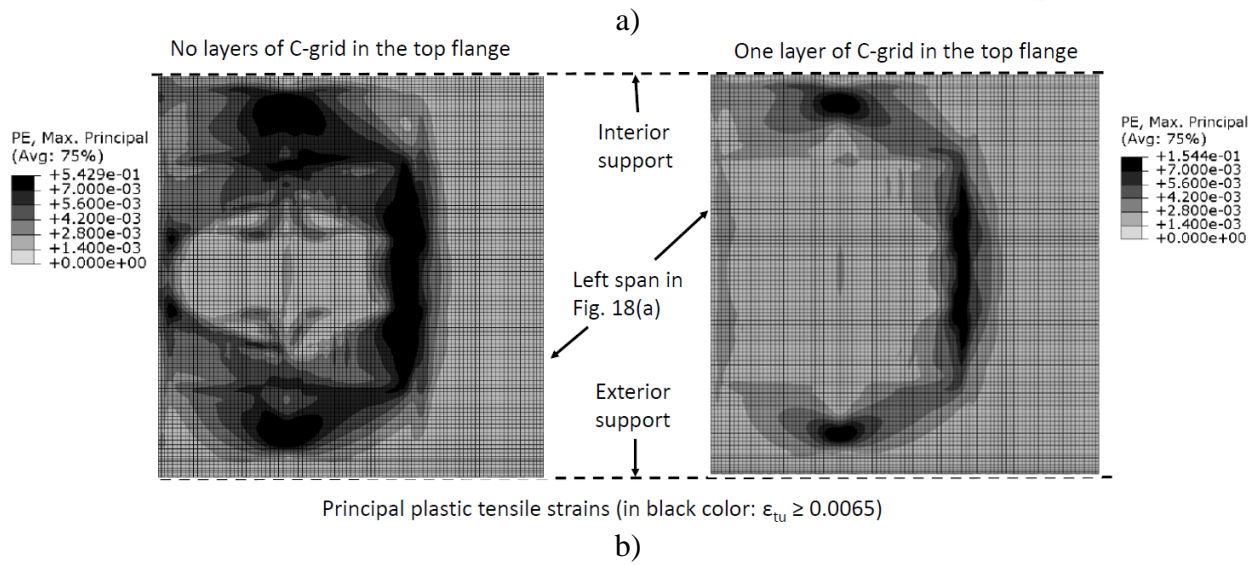1002   a)   b)
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Load position c₃

Shear crack

Mid-width Line/
Axis of Symmetry

PE, Max. Principal
(Avg: 75%)
+6.301e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Flexural crack

Principal plastic tensile
strain at ultimate
load (in black color: $\varepsilon_{tu} \geq 0.0065$).

1023
1024

a)

No layers of C-grid in the top flange

One layer of C-grid in the top flange

PE, Max. Principal
(Avg: 75%)
+5.429e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Interior
support

Left span in
Fig. 18(a)

Exterior
support

PE, Max. Principal
(Avg: 75%)
+1.544e-01
+7.000e-03
+5.600e-03
+4.200e-03
+2.800e-03
+1.400e-03
+0.000e+00

Principal plastic tensile strains (in black color: $\varepsilon_{tu} \geq 0.0065$)

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b)

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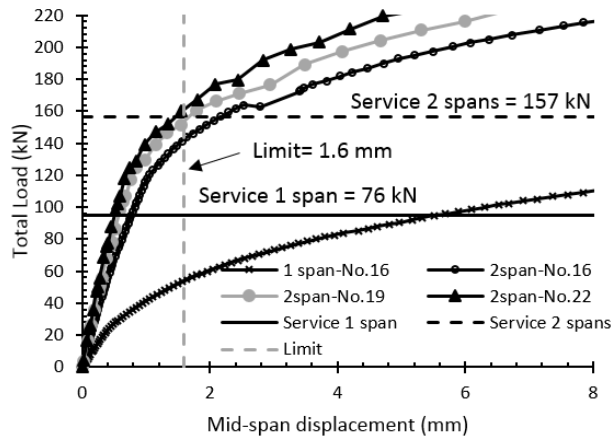
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Table 1 Recommendations for UHPC deck panel

Recommendations		Reason/Comment
Description	Value	
Panel Thickness (mm)	132	To be consistent with predominant existing grid deck thicknesses
Max. Panel weight (kN/m^2)	0.96	Calculated based on stringer reactions
Max. Span length (m)	1.27	It covers the majority of existing steel grid deck spans
Continuity	3+	Min. of three spans or simple span made continuous
Stringer	W410×54	Most common stringer

Table 2 Parameters for concrete damage plasticity model

Parameter	Value
Dilation angle (degrees)	45
Eccentricity	0.10
$\dfrac{\sigma_{bo}}{\sigma_{co}}$	1.77
K	0.67
Viscosity parameter	0

Table 3 Comparison of peak loads obtained numerically and experimentally

Specimen Description	Peak Loads (kN)		Ratio = P_{Num}/P_{Exp}
	Numerical	Experimental	
1T1S	168	182	0.92
1T2S	248	246	1.01
4T1S	387	377	1.03
3T2S	686	656	1.05
			Average = 1.00
			COV = 0.06

Table 4 Computed stress versus developable stress

Type	Size	Transverse			Longitudinal		
		f_{fe} (MPa)	$f_{computed}$ (MPa)	$f_{fe}/f_{computed}$	f_{fe} (MPa)	$f_{computed}$ (MPa)	$f_{fe}/f_{computed}$
	No.10				852	690	1.24
	No.13	1064	830	1.28			
GFRP	No.16	926	758	1.22			
	No.19	822	634	1.30			
	No. 22	750	482	1.55			