

Numerical Forensic Model for the Diagnosis of a Full-Scale RC Floor

Abstract

The paper presents the results of an investigation on the diagnosis and assessment of a full-scale reinforced concrete floor utilizing a 3-D forensic model developed in the framework of plasticity-damage approach. Despite the advancement in nonlinear finite element formulations and models, there is a need to verify models on nontrivial challenging structures. Various standards on strengthening existing structures consider numerical diagnosis as a major stage involving safety and economical aspects. Accordingly, model validity is a major issue that should preferably be examined against realistic large-scale tests. This was done in this study by investigating a one-story joist floor with wide shallow beams supported on columns. The surveyed cracking patterns on the entire top side of the floor were reproduced by the forensic model to a reasonable degree in terms of orientation and general location. Concrete principal plastic tensile strain was shown to be a good indirect indicator of cracking patterns. However, identifying the underlying reasons of major cracks in the floor required correlating with other key field parameters including deflections, and internal moments. Therefore, the ability of the forensic model to reproduce the surveyed damage state of the floor provided a positive indication on the material models, spatial representation, and parameter selection. Such models can be used as forensic tools for assessing the existing conditions as required by various standards and codes.

Keywords

nonlinear finite element; reinforced concrete; forensic; cracking; numerical model; diagnosis; existing structures.

Ahmed B. Shuraim

Associate Professor, College of Engineering (Civil), King Saud University, Riyadh, Saudi Arabia

Received 30 Mar 2012;
In revised form 26 Apr 2012

* Author email: ashuraim@ksu.edu.sa

1 INTRODUCTION

Many existing Reinforced Concrete (RC) structures show a number of situations where the load carrying capacity of a structure in service may need to be increased, like where there is a change of loading of use, or where the structure has been damaged. With reference to

6 a deficient RC structure, three stages of assessment have been outlined by some structural
7 design guidelines [1, 2] investigation of current conditions, diagnosis, selection of appropriate
8 solution and strengthening design using FRP materials [11]. Because proper diagnosis has
9 strong implications on various aspects of the structure including strength, strengthening cost,
10 safety, etc., robust reliable 3-D forensic models are crucial for real life full- three dimensional
11 structures.

12 Nonlinear finite element (NLFE) for reinforced concrete structures has many applications
13 [4, 5] including: providing an insight into the behavior; predicting possible modes of failure;
14 corroborating the experimental findings; and extending the findings to locations where mea-
15 surements are not available. From the perspective of forensic studies in relation to damaged
16 or ageing structures, NLFE can be can be of value to assess the safety and integrity of existing
17 structures; identify causes of deficiencies; and to assess the behavior expected from retrofitted
18 structures. However, NLFE models comprise various theories, elements and procedures where
19 some limitations, shortcomings and complexity were reported [34, 35].

20 The complexity of finite element is in part due to the complexity of the concrete physical
21 behavior, being largely determined by the properties and interaction among its constituent
22 materials. To incorporate these experimentally-observed features into finite element, several
23 constitutive modeling theories have been proposed, including: nonlinear elasticity, plasticity,
24 and damage mechanics, among others [14, 18, 26]. Coupled damage-plasticity formulation
25 can provide a better representation of salient concrete features [18, 26], which is among the
26 concrete models incorporated in ABAQUS .

27 Numerical modeling of cracking in concrete usually involves either discrete or smeared
28 cracking approaches [9, 16, 28]. Discrete approach represents a crack as a geometrical disconti-
29 nuity and can be treated in the framework of fracture mechanics concepts. The main drawback
30 is that one needs to know location of an expected crack and the direction along which it prop-
31 agates [28]. On the contrary, smeared crack through damage mechanics offers the essential
32 advantage of predicting the location of this critical flaw. To circumvent mesh-sensitivity and
33 numerical divergence, an energy criterion can be employed instead of strength failure criterion.
34 In ABAQUS, fictitious crack model introduced by Hillerborg (1985)[21] can be employed to
35 alleviate the mesh-sensitivity. Whatever the chosen approach, numerical models need to be
36 validated not only on the basis of small members with trivial geometry but rather on the basis
37 of more challenging reinforced concrete structures [9].

38 This study developed a forensic model on the basis of ABAQUS formulations and numerous
39 material models from the literature. The model capability was demonstrated over a full-scale
40 building floor for the purpose of assessing its adequacy in predicting the overall behavior in
41 terms of deformations and cracking patterns. Furthermore, the correlation between cracking
42 and internal distribution of moments was established at critical sections, giving the background
43 for the newly proposed design procedure [29, 30]. The results showed a successful corroboration
44 with test results indicating that such models can be used as forensic models for assessing the
45 existing conditions as required by various standards and codes.

2 DESCRIPTION OF THE FULL-SCALE REINFORCED CONCRETE JOIST FLOOR

The plan of WSB floor shown in Fig. 1 is for a full-scale reinforced concrete joist floor, designed, constructed and monitored for more than 10 years within the structural laboratories at King Saud University [29, 30]. Fig. 2 is a photo of the floor showing loading and members. The floor consists of wide-shallow beams, one-way joists, a portion of solid slab, and narrow columns that were within the typical widths of masonry walls. Dimensions, main reinforcement, and material properties are given in Table 1.

Testing of the roof slab of the prototype building was done in three stages. Stage 1 was the application of floor self-weight (SW) through form-removal. In Stage 2, a line load of 10.0 kN/m was applied on Joist J19 (WJ19) while in Stage 3, a uniform load (U) of 3.5 kN/m² over the entire floor was applied. Concrete blocks were used for load applications. Stages 2 and 3 were applied in a set of mini-intervals where deflections, strains and cracking patterns were collected during and after each interval. Net and accumulated deflections and strains due to immediate load effects were assembled for each stage. The instrumentation plan involves measurement of strains and deflections using various techniques.

The state of cracking on this floor was marked and photographed under all loading stages and throughout their sustained conditions. Examples of the cracked states in the WSB floor were depicted by photos presented in Fig. 3 through Fig. 6. The paths of these cracks were documented by linear measurements of their coordinates to allow plotting them with the numerical results.

Table 1 Floor member dimensions, reinforcement and material properties.

Member	Section dim b x h mm	Longitudinal rebars No of bars x diameter in mm	Ties/stirrups	MPa	MPa
B1, B2	250 x 800	Top 4 φ 16, 11 φ 16 ⁽¹⁾ Bot 7 φ 16	8@100 mm (4-Leg) at B1 ends	25	511
B3 , B4	250 x 1200	Top 5 φ 16, 17 φ 16 ⁽¹⁾ Bot 9 φ 16	8@100 mm (4-Leg) at B3 ends		
DB1, DB2	600 x 200	Top 2 φ 16	8@100 mm (2-Leg)		
SS15	h=150	Top φ 12@550 ⁽²⁾ Bot φ 16@400 ⁽³⁾	None		499 511
J2-J22	250 x 150	Top 1 φ 12 Bot 2 φ 14	None		499 505
J1& J23 ⁽⁸⁾	250 x 200	Top 1 φ 12 Bot 2 φ 14	None		499 505
C1,C4,C8 ⁽⁹⁾	200x 500	8 φ 16	8@200 mm (4-Leg)	28	511
C2	200x 600	8 φ 16	8@200 mm (4-Leg)		511
C3, C6,C7, C9	200x 400	8 φ 14	8@200 mm (4-Leg)		505
C5	200 x 800	10 φ 16	8@200 mm (5-Leg)		511

Notes :(1) rebars at the interior support; (2) in the short direction only; (3) in short and long directions; (4) in the model, J14 was modeled as J1 and J23; (5) orientation as shown in Fig. 1

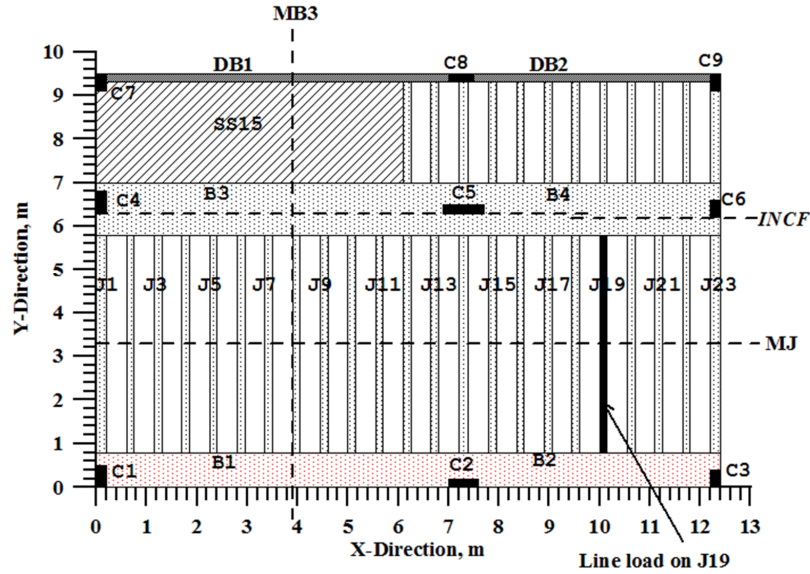


Figure 1 Floor Plan layout, showing member designations

66 **3 SPATIAL AND MATERIAL ASSUMPTIONS OF THE FORENSIC MODEL**

67 Numerical models developed to simulate structure behavior may differ based on the choices
 68 made in their spatial modeling, material modeling, element characteristics, and solution pro-
 69 cedures. For WSB floor shown in Figures 1 and 2, the nature of the problem is complex
 70 and requires three-dimensional for geometrical and material considerations. The complexity
 71 may be induced by non-uniformity in floor thickness, existence of columns with eccentricities,
 72 different rebar distributions, partial column-floor connections, stirrup arrangements, triaxial
 73 effects, and different potential modes of failure. Spatially, concrete can be modeled with a
 74 variety of 2-D and 3-D elements. In this study, solid 3-D elements (C3D8) were selected for



Figure 2 A photo of the tested floor showing loading and part of the geometry

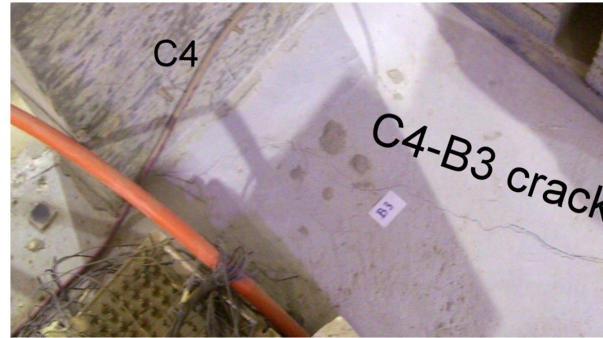


Figure 3 Photographic documentation of crack patterns around C4

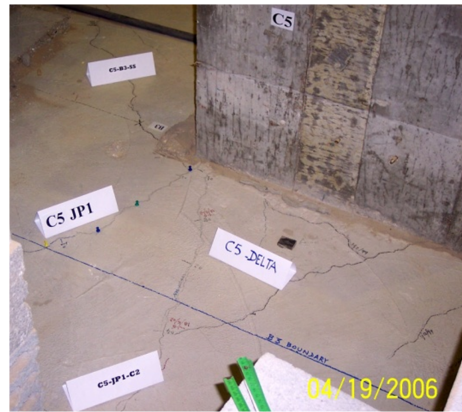


Figure 4 Photographic documentation of crack patterns around C5-long side

75 concrete while a bi-linear elastic-plastic model was chosen for reinforcing steel and represented
 76 in space via truss elements (T3D2). The resulting spatial models for the floor concrete and
 77 rebars are presented in Fig. 7 and Fig. 8.

78 In this study, ABAQUS damage-plasticity model was chosen for modelling concrete be-
 79 haviour. The post-peak in tension was represented by Hillerborg Fictitious Crack Model
 80 (HFCM) (Hillerborg 1985)[21], which is characterized by three key parameters: the specific
 81 energy dissipated after the localization in the cracked region; the tensile strength in direct
 82 tension; and, the shape assumed for the softening branch. The energy criterion has been em-
 83 ployed instead of strength failure criterion in order to minimize the superficial dependence on
 84 the chosen finite element.

85 3.1 Material parameters

86 The input parameters for the damage-plasticity model of concrete include uniaxial compression
 87 curve, uniaxial tensile strength, Poisson's ratio, modulus of elasticity, dilatancy angle, and
 88 fracture energy. These parameters were defined as per available test data as well as from
 89 appropriate constitutive models from the technical literature [29–32]. The fracture energy,



Figure 5 Photographic documentation of crack patterns on C5-C8 path

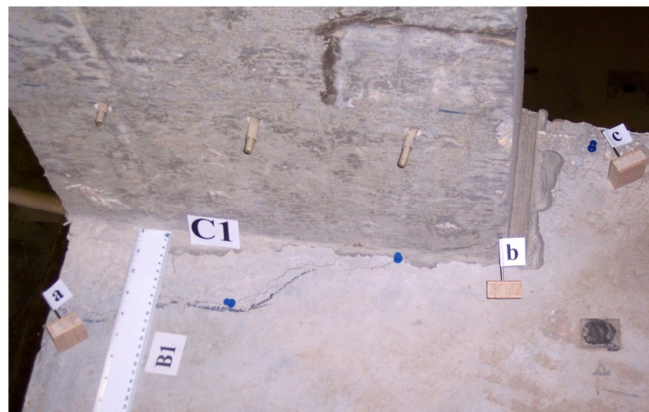


Figure 6 Photographic documentation of crack patterns around C1

90 G_f , was expressed by Eq. (1) [3]

$$Gf = 110 \left(\frac{f'_c}{10} \right)^{0.18} \quad (1)$$

91 Direct tensile strength, f'_t (MPa) was taken as

$$f'_t = 0.25 \sqrt{f'_c} \quad (2)$$

92 Post-cracking tensile curve for concrete shown in Fig. 9 a was defined by Eq. (3) [25],
93 where n is a coefficient taken as 2.

$$f_t = f'_t = \left(1 - \frac{x}{d_0} \right)^n ; d_0 = \frac{Gf(n+1)}{f'_t} \quad (3)$$

94 For uniaxial compression stress-strain curve, the following expression was adopted following

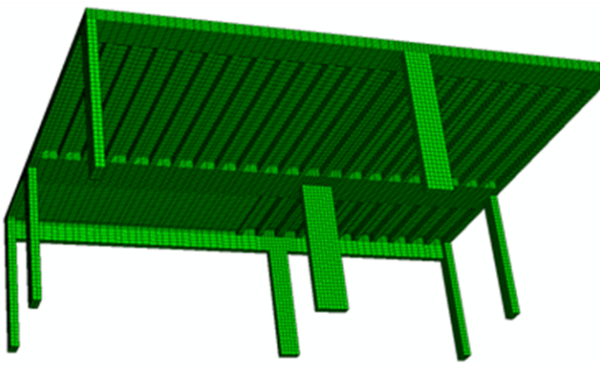


Figure 7 Concrete solid elements and mesh of WSB model.

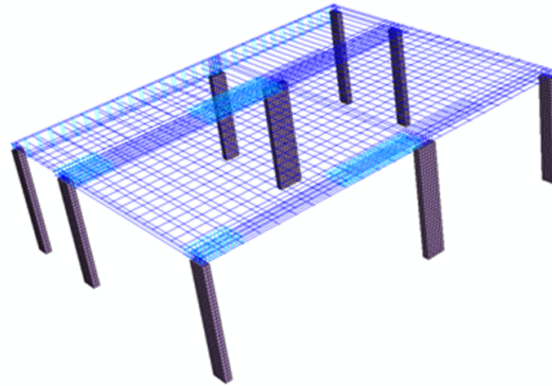


Figure 8 Rebars layout in the WSB model.

95 Carreira, and Chu (1985)[13]:

$$\frac{f_c}{f'_c} = \frac{\beta \frac{\epsilon}{\epsilon_0}}{\beta - 1 + \left(\frac{\epsilon}{\epsilon_0}\right)^\beta}; \text{ where } \beta = \frac{1}{1 - \frac{f'_c}{\epsilon_0 E_c}} \quad (4)$$

96 Where the strain at maximum stress was defined by $\epsilon_0 = 0.001648 + 0.0000165, f'_c$, Ahmed
 97 (1981)[10]; and the modulus of elasticity for concrete, E_c (MPa) was given by ACI-363 (1997)
 98 [7] $E_c = 3000\sqrt{f'_c} + 6900$. The plastic part of the compression stress-strain curve is shown in
 99 Fig. 9 b. The resulting model in its deformed shape along with the contour of maximum
 100 principal tensile strains in concrete is shown in Fig. 10.

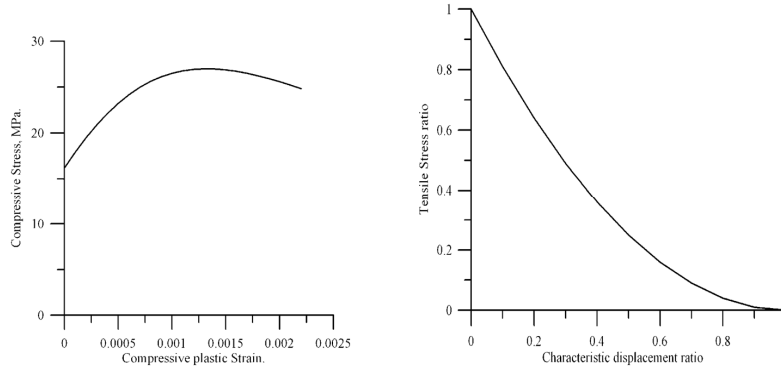


Figure 9 Typical stress-strain curves of concrete; (a) compressive plastic strain vs. stress, (b) post-peak tensile stress vs. displacement

101 **4 FLOOR PHYSICAL CRACKING VERSUS CONCRETE TENSILE STRAINS**

102 Generally speaking, concrete cracking is an elusive phenomenon of complicated nature in the
 103 sense of the underlying causes, the significance, and the overall pattern. Cracks may indicate

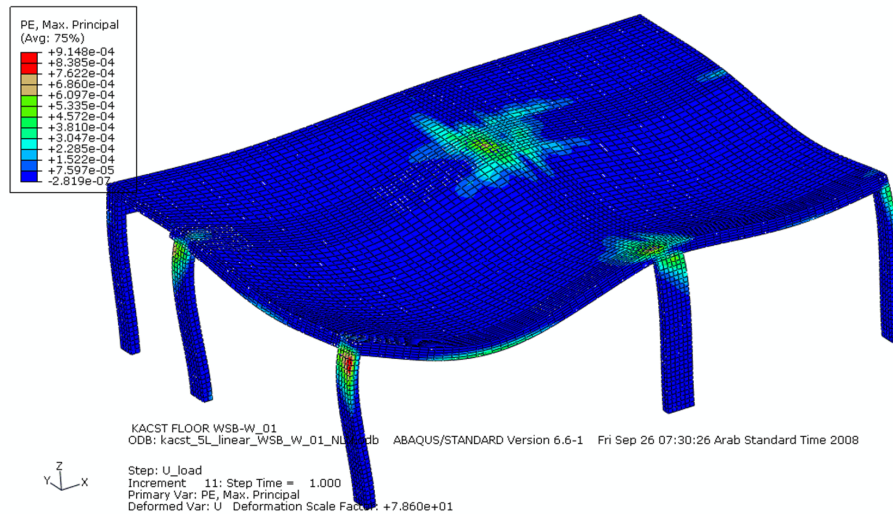


Figure 10 Deformed shape of the floor showing the contour of maximum principal tensile strains in concrete

104 significant structural distress or a lack of durability. They may also represent the total extent
 105 of the damage, or they may point to problems of greater magnitude [6]. In many instances,
 106 cracking may be attributed to a combination of stresses due to applied load and restraint of
 107 volume change. The path a crack takes is also affected by the concrete composite properties
 108 where the aggregate and the interfaces play very important role in the failure process [17].

109 However, when surveying the damage of an existing structure, the cracking patterns can be
 110 detected and documented with reasonable efforts where no particular setup or instrumentations
 111 are required. A key quality of a good forensic model is its ability to reproduce existing physical
 112 cracks and provide some insight into the underlying causes. The smeared crack approach does
 113 not produce actual physical cracks, however, strains or stresses can usually be interpreted,
 114 though not directly, to show the potential crack patterns [12, 15, 16, 19, 23, 27, 31–33].

115 In ABAQUS, cracking is assumed to initiate at points where the tensile equivalent plastic
 116 strain is greater than zero, and the maximum principal plastic strain is positive [22]. The
 117 direction of the vector normal to the crack plane is assumed to be parallel to the direction of
 118 the maximum principal plastic strain. No information can be obtained regarding crack spacing
 119 in smeared crack formulation and it may not be a significant issue when investigating large
 120 structures at macroscopic level. For the floor of this study, physical major cracks on the top
 121 side of the floor were mapped and plotted to a reasonable scale in Fig. 11 through Fig. 14.
 122 For direct comparison with the forensic model results, the maximum plastic principal tensile
 123 strains (PPTS) were superimposed on the same figures.

124 4.1 Areas around floor corner columns

125 The corner columns in this floor had some framing differences affected by the panels they are
 126 carrying and the size of beams. Column C1 is part of the largest panel while C3 is in a medium

127 panel and both are attached to wide shallow beams. On the other hand, C7 is attached to a
128 drop beam carrying solid slab while C9 is part of a joist panels and framing into a drop beam.
129 For the areas around the latter two columns, there is a good correlation between physical
130 observation and model prediction since these areas did not exhibit any physical cracking and
131 no potential of cracking as demonstrated by the lack of noticeable PPTS symbols.

132 On the other hand, column C1 was surrounded by circumference crack that was reproduced
133 qualitatively through PPTS symbols. This crack was apparently induced by the presence of
134 biaxial negative moment about the x and y-axes. Finally, the crack around C3 was reasonably
135 predicted in a way that is consistent qualitatively with PPTS and in general agreement with
136 the expected crack orientation from the biaxial moment behavior.

137 4.2 Areas around floor side columns

138 The floor side columns in this floor include C2, C4, C6, and C8 where the latter is framed into
139 a drop beam while the remaining three were framed into wide shallow beams. Two of these
140 areas are to be discussed next while the other two are to be covered in a subsequent section.

141 4.2.1 C2 area

142 Column C2 supports unequal spans (B1 and B2), along its major axis about y-axis. From
143 the floor layout, C2 had the potential also of carrying some moment about its minor x-axis.
144 The area around C2 within the beam width showed several cracks oriented primarily in the
145 y-direction as shown in Fig. 13. From the forensic model, the overall PPTS intensity and
146 orientation were consistent with physical cracks orientations and general location. Follow-
147 ing the PPTS symbol convention, the simulated crack should travel in the y-direction in a
148 perpendicular direction to the symbols which were primarily parallel to x-axis.

149 4.2.2 C8 area

150 In comparison with C2, the PPTS in the area around C8 showed shorter arrow length, and
151 less intensity indicating that cracking level at C8 should be less than C2. The floor showed a
152 visible crack emerging from the corner of column C8 and another crack crossing the adjacent
153 drop beam (DB1). Having the cracks on the left side of C8 was anticipated because the span
154 of DB1 was larger than the span of DB2. The crack designated C5-C8 extended all over the
155 panel was reproduced by the forensic model as indicated by the PPTS symbols in Fig. 11 and
156 Fig. 12.

157 4.3 Areas around the interior column

158 The state of cracking in the area around column C5 was influenced by high intensity of mul-
159 ti-axial moments and shear forces as well as the rectangularity of the column cross-sectional
160 shape. Mapped physical cracks as shown in Fig. 15 revealed that most of the cracks emerged
161 from the vicinities around the column corners. Because of the high aspect ratio of the column
162 section; the short sides became the dominating source of multi-directional cracking. It is to
163 be noted that the overall pattern of the local cracks around C5 resembled to some extent the

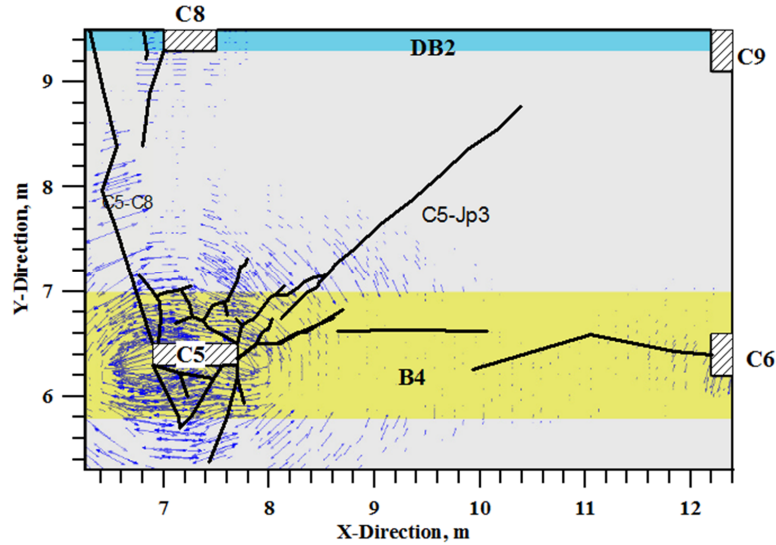


Figure 11 Mapped major cracks along with PPTS in panel C5-C6-C8-C9

164 state of cracking in a flat plate system on rectangular columns prior to punching shear failure,
 165 comprising tangential and diagonal cracks.

166 Column rectangularity effect on punching shear was recognized by previous studies [8, 24]
 167 where the critical shear strength was found to decrease as the aspect ratio increases, such as:

$$v_c = \frac{\left(2 + 3 \frac{C_s}{C_l}\right)}{12} \sqrt{f'_c} \text{ (mpa)} \quad (5)$$

168 where C_s is the short side of the column and C_l is the longer side and f'_c is the concrete
 169 strength. The rectangularity effect in this floor was demonstrated by the concentration of
 170 cracking around the short sides of the column. This cracking behavior provides a corroboration
 171 of previous studies on the effect of column rectangularity on punching shear.

172 Fig. 15 shows that the overall PPTS intensity and orientation around column C5 is in
 173 support of the physical cracks orientations and general location. From this symbolic repre-
 174 sentation, potential cracks can be viewed to propagate in a perpendicular direction to the
 175 two-arrow lines. Fig. 15 shows that the major diagonal cracks to the neighboring panels were
 176 clearly predicted except for Jp2 where no physical crack was observed. In general, the forensic
 177 model has captured the essence of the behavior to a large extent where the areas covered by
 178 PPTS mirrored the areas that were subjected to physical cracks on the actual floor.

179 **5 DESIGN CONSIDERATIONS: CORRELATION OF INTERNAL MOMENTS WITH**
 180 **CRACKING**

181 The complexity of internal moment distribution in concrete slabs has been well recognized;
 182 accordingly, codes of practice prescribe certain procedures for approximating the distribution

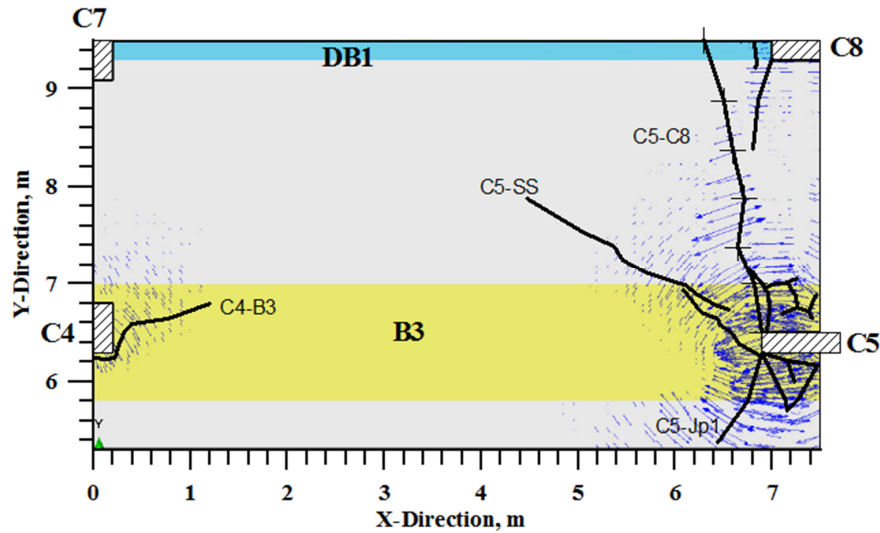


Figure 12 Mapped major cracks along with PPTS in panel C4-C5-C7-C8

183 for design purposes. For WSB floors with one-way joist system, the conventional distribution
 184 is questionable [29, 30] and it would be appropriate to utilize the forensic model and the
 185 experimental findings to verify the internal moment distribution in this floor by correlating
 186 with physical cracks and PPTS.

187 When using solid elements and discrete truss rebar elements, the internal moments and
 188 forces are not readily available from the standard output of ABAQUS. Thus, the moments were
 189 developed on the basis of the nodal forces and nodal coordinates. A computer post processing
 190 program was written for this purpose employing the following fundamental equations with
 191 notation as shown in Fig. 16.

192 Using vector notations,

$$\vec{M} = \sum \vec{r}_n \times \vec{F}_n = M_x \vec{i} + M_y \vec{j} + M_z \vec{k} \quad (6)$$

193 Where \vec{r}_n is the position vector from the center of the section to a given node, n.

$$\vec{r}_n = (\Delta_x) \vec{i} + (\Delta_y) \vec{j} + (\Delta_z) \vec{k} \quad (7)$$

194 where x_0 , y_0 , and z_0 are the coordinates for the section centroids; x_n , y_n , and z_n are the
 195 coordinates for a given node n . similarly, the nodal forces at node n may be written as:

$$\vec{F}_n = (F_{xn}) \vec{i} + (F_{yn}) \vec{j} + (F_{zn}) \vec{k} \quad (8)$$

196 The moments about x and y axes may be obtained by implementing the cross product of
 197 Eq. (6):

$$M_x = \sum_n (y_n - y_0) F_z - (z_n - z_0) F_y$$

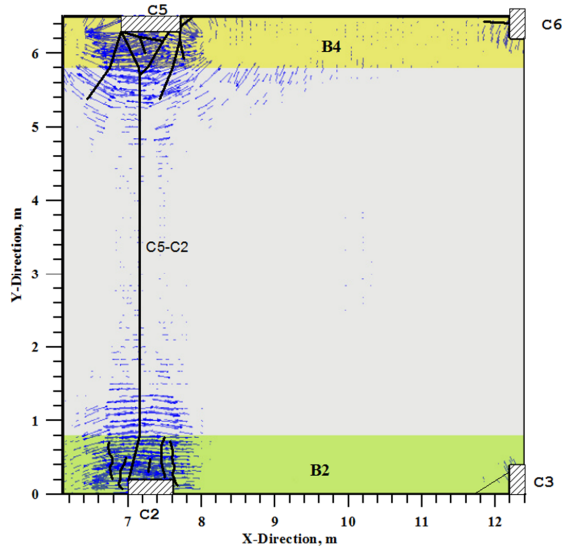


Figure 13 Mapped major cracks along with PPTS in panel C2-C3-C5-C6

$$M_y = \sum_n (z_n - z_0) F_x - (x_n - x_0) F_z \quad (9)$$

198 The moment diagrams about y-axis and x-axis over the interior shallow wide member beam
 199 B3-B4 are shown in Fig. 17 and it is of interest to examine the correlation of these moments
 200 with physical cracks. Under the sign convention used in this study, negative moment exceeding
 201 the cracking moment about x-axis (M_x) should produce top cracking along x-axis (i.e along
 202 the beam) while negative moment about y-axis (M_y) should produce cracking along y-axis (i.e
 203 across the beam). Their relative values can influence the direction and path of the crack
 204 on the beam.

205 First, Fig. 17 reveals that the exterior end of B3 is subjected to a higher intensity of (M_x)
 206 moment coupled with a low intensity of (M_y); this distribution favors cracks in the x-direction
 207 which was consistent with the observed crack (C4-B3) in Fig. 12. Second, the middle part
 208 of B3 has positive moment and negligible negative moment (M_x) suggesting that no cracks
 209 should be expected on the top side of B3. As expected, the floor did not exhibit cracks on the
 210 middle portion of B3 corroborating the results from the forensic model.

211 Third, for the middle and exterior portion of B4, the figure shows that the moment (M_x) is
 212 negative with appreciable value in comparison to its counterpart on B3; moreover, the moment
 213 (M_y) is either positive or of negligible negative value. This situation calls for cracking along
 214 B4, and this was the observed pattern. Finally, Fig. 17 shows that the interior ends of B3
 215 and B4 were subjected to high negative moments about both axes inducing multi-directional
 216 cracking, as observed.

217 Thus, in addition to diagnosis of the existing floor conditions, the information revealed
 218 about the internal distribution of moments indicated that the common simplified procedure
 219 for one-way joist floor system which assumes a uniform intensity of (M_x) was unrealistic and

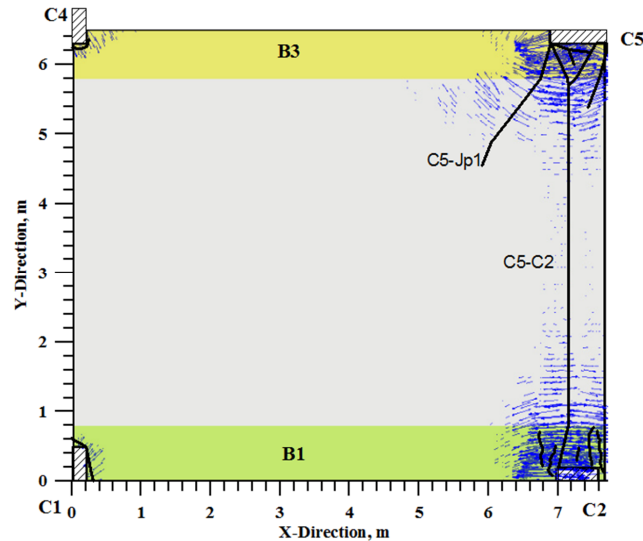


Figure 14 Mapped major cracks along with PPTS in panel C1-C2-C4-C5

220 contradicted by the forgoing profile. This discrepancy between the assumption and the actual
 221 distribution was investigated by [30] and a new design procedure has been proposed.

222 6 FLOOR DEFLECTIONS: VERIFICATION AND INTERPRETATIONS

223 For further corroboration of the analytical forensic model with test results, vertical deflection
 224 profiles along two lines in the floor were examined. Deflections are among the key behavioral
 225 aspects and the ability of a forensic model to produce a reliable assessment of such large scale
 226 test is of a great value in terms of validation and behavior interpretation. The deflections
 227 profiles along line MJ and line MB3 are plotted in Fig. 18 and Fig. 19, respectively. It
 228 can be observed that the forensic model produced deflections that were in good agreement
 229 with test results for the three loading stages. The three experimental measuring schemes
 230 (dial gauges (DG), LVDT, and surveying technique (SVY)) employed in this study produced
 231 very consistent results with some expected scatter in readings. The agreement in trend and
 232 magnitude between experimental and the forensic model had implication on mutual validation
 233 of test measurements and numerical predictions.

234 Moreover, deflection results can be interpreted to shed some light on the long crack from C2
 235 to C5 and that from C5 to C8, recognizing the role of edge members rigidities on the slab panel
 236 behavior. For a typical rectangular slab panel, the load transfer and distribution of moments
 237 in the two-directions is governed by the rigidity of edge members and panel continuity. This
 238 principle remains true to some extent for rectangular joist panel; although, joist panels have
 239 more complexity due to the interaction between unidirectional ribs and the top slab panel that
 240 represents the flange.

241 For the panel bounded by columns C1, C2, C4, and C5, the portion of the joist panel

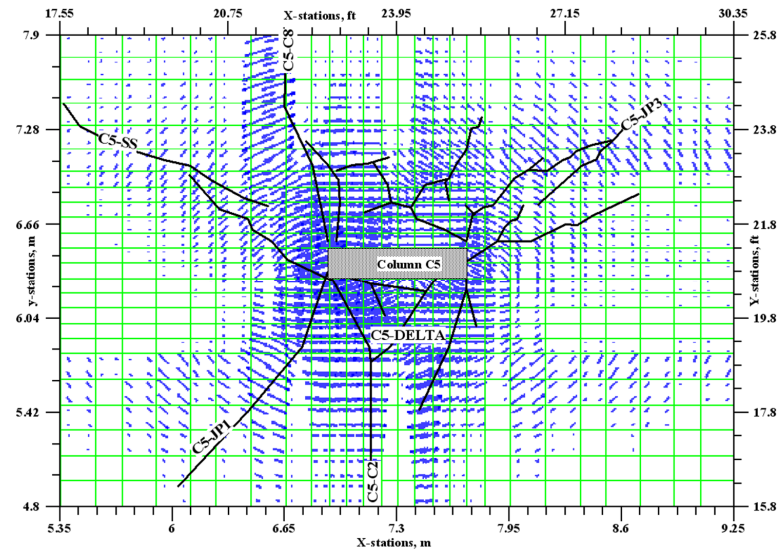


Figure 15 Mapped major cracks around column C5 along with PPTS

242 connecting column C2 to C5 represents a de facto supporting member due to its high rigidity
 243 as influenced of the two columns. This high rigidity can be interpreted from the deflection
 244 profile along MJ line (Fig. 18) where the panel edges are marked by lower relative deflection
 245 in comparison to mid-panel. It follows that a negative moment will form about y-axis on line
 246 C2-C5, and since the flange is weak in resisting the moment, cracking has to take place. The
 247 panel behavior may also be viewed in terms of compatibility requirements where the joist panel
 248 needs to deform in a compatible way with the beams B1 and B3.

249 From the forgoing discussion, deflections were properly produced by the forensic model
 250 and the results were shown to correlate with cracking along line C2-C5-C8. Even though some
 251 concerns have been raised about NLFE, forensic models in the framework of plasticity-damage
 252 approach and Hillerborg Fictitious Crack Model (HFCM) as supplemented by appropriate
 253 constitutive expressions provided a powerful, accurate platform that can be used for forensic
 254 assessment of distressed or failed reinforced concrete structures.

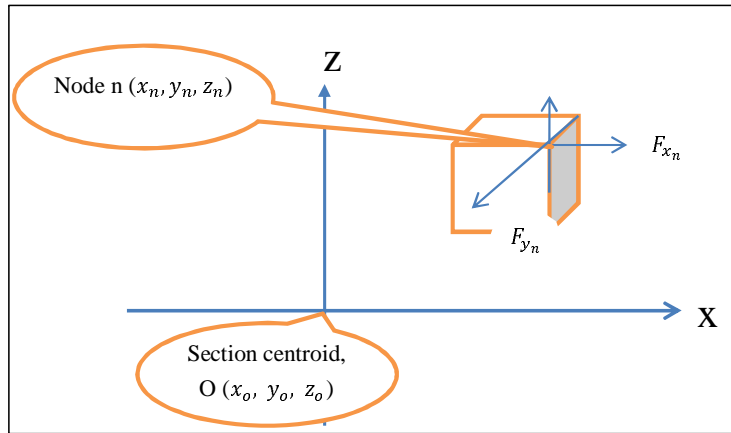


Figure 16 Computation of section moments from solid nodal forces

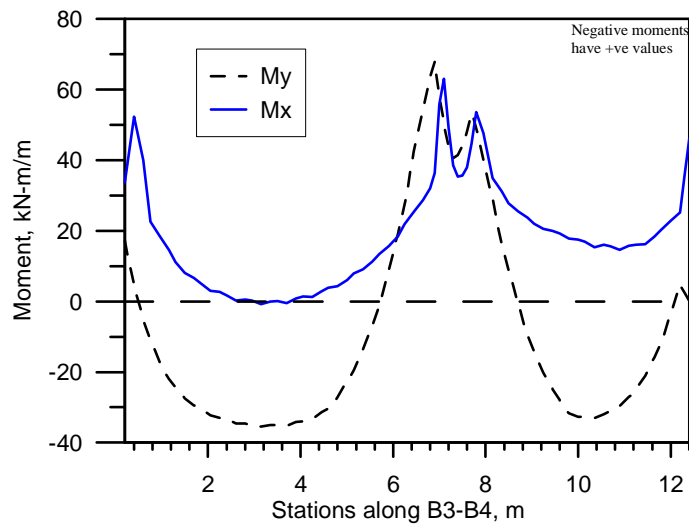


Figure 17 Moment profiles about x and y-axes along beams B3 and B4

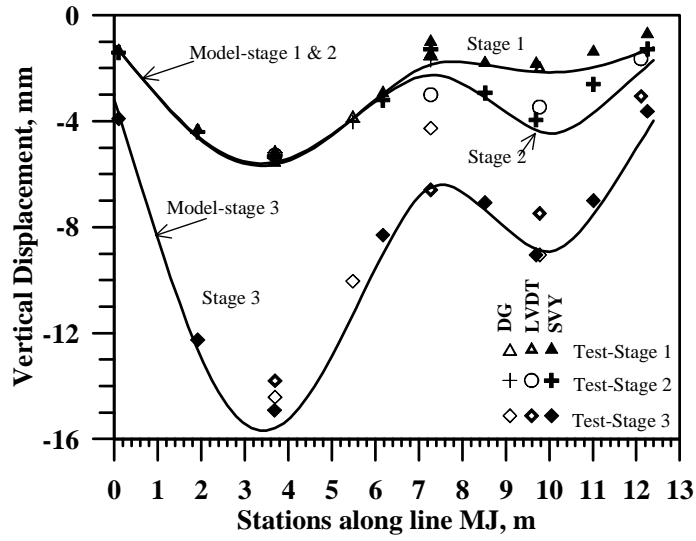


Figure 18 Deflection profile along line MJ in WSB model and tested floor

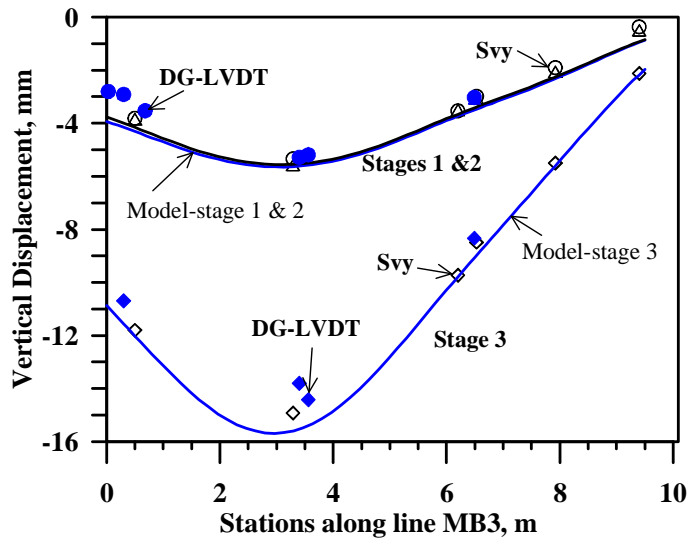


Figure 19 Deflection profile along line MB3 in WSB model and tested floor

7 SUMMARY AND CONCLUSIONS

This paper presented the results of an investigation on the diagnosis and assessment of an existing structure employing a nonlinear forensic model. Various standards consider numerical diagnosis as a major stage in the assessment and strengthening of existing structures based on which several decisions involving safety and economical aspects are made. Accordingly, an integral part of the methodology is to verify the forensic model reliability through comparison with realistic large-scale tests. This was done in this study by investigating one-story floor of joist-construction style with hidden wide shallow beams supported on elongated columns. Based on this investigation, the following points can be made:

1. The forensic model and the experiment agreed on the behavior at macroscopic level in terms of deflections and cracking patterns. Concrete principal plastic tensile strain (PPTS) was shown to be a good indirect indicator of cracking patterns in terms of cracking location and direction through symbolic representation. However, identifying the underlying reasons of major cracks in the floor required correlating with other key field parameters including deflections, and internal moments.
2. The ability of the forensic model to reproduce the surveyed damage state of the floor provided a positive indication on the material models, spatial representation, and parameter selection. Spatially, well-balanced mesh of concrete and rebar elements was shown to be sufficient to provide reasonably accurate information. Having extremely fine mesh as usually performed on individual members is not practical neither seemed necessary for 3-D buildings. The plasticity-damage approach combined with HFCM and supplemented by appropriate constitutive expressions provided robust model.
3. The distribution of internal moments about the longitudinal axis of the wide shallow beams (M_x) was of variable nature and consistent with the observed cracking patterns. This finding confirmed the need to improve the design methodology of one-way joist systems on wide shallow beams, as proposed elsewhere [30].

Acknowledgments The author gratefully acknowledges the support provided by King AbdulAziz City for Science & Technology under Grant No. AR-16-66 in the experimental work. Any opinions, findings, and conclusions expressed in this paper are those of the writers and do not necessarily reflect the views of the sponsor.

References

- [1] FIB Bulletin 14. Externally bonded frp reinforcement for rc structures. Technical report, FIB, Lausanne, Switzerland, 2001.
- [2] ACI-440 2R. *Design and construction of externally bonded FRP systems for strengthening concrete structures*. Farmington Hills, MI, 2008.
- [3] Fib Bulletin 42. Constitutive modelling of high strength / high performance concrete. state-of-art report by task group 8.2. Technical report, International Federation for Structural Concrete, Lausanne, Switzerland, 2008.

- 292 [4] ASCE-ACI Committee 447. State of the art report on finite element analysis of reinforced concrete, 1982.
- 293 [5] ASCE-ACI Committee 447. Finite element analysis of reinforced concrete structures ii, 1993.
- 294 [6] ACI-224.1R. *Causes, evaluation, and repair of cracks in concrete structures*. Farmington Hills, MI, 26 edition, 2007.
- 295 [7] ACI-363R-92. *State of the art report on high strength concrete*. Farmington Hills, MI, 55 edition, 1997.
- 296 [8] ACI-426. "The shear strength of reinforced concrete members" Part 4 of ACI manual of concrete practice. Farmington
297 Hills, MI, 1987.
- 298 [9] ACI-446. *Finite Element Analysis of Fracture in Concrete Structures: State-of-the-Art*. Farmington Hills, MI, 1997.
- 299 [10] S. H. Ahmed. *Properties of confined concrete subjected to static and dynamic loading*. PhD thesis, University of
300 Illinois at Chicago, 1981.
- 301 [11] F. Bencardino, L. Rizzuti, and G. Spadea. Strengthening/retrofitting of an rc structure with bonded cfrp laminates:
302 reliability evaluation using different guidelines. *FRPRCS-8*, July 16-18 2007.
- 303 [12] P. Bhatt and M. Abdel Kader. Prediction of shear strength of reinforced concrete beams by nonlinear finite element
304 analysis. *Computers and Structures*, 68:139-155, 1998.
- 305 [13] D. J. Carreira and K. Chu. Stress-strain relationship for plain concrete in compression. *ACI Journal*, 82:797-804,
306 1985.
- 307 [14] W. F. Chen and D. J. Han. *Plasticity for Structural Engineers*. Springer-Verlag, New York, 1988.
- 308 [15] D. Coronelli and M. G. Mulas. Modeling of shear behavior in reinforced concrete beams. *ACI Structural Journal*,
309 103(3):372-382, 2006.
- 310 [16] R. de Borst. Fracture in quasi-brittle materials: a review of continuum damage-based approaches. *Engineering*
311 *Fracture Mechanics*, 69:95-112, 2002.
- 312 [17] G. Giaccio and R. Zerbino. Failure mechanism of concrete: combined effects of coarse aggregates and strength level.
313 *Advanced Cement Based Materials*, 7(2):41-48, 1998.
- 314 [18] P. Grassl and M. Jirasek. Damage-plastic model for concrete failure. *International Journal of Solids and Structures*,
315 43:7166-7196, 2006.
- 316 [19] A. D. Hanganu, E. Onate, and A. H. Barbat. A finite element methodology for local/global damage evaluation in
317 civil engineering structures. *Computers and Structures*, 80:1667-1687, 2002.
- 318 [20] Hibbitt, Karlsson & Sorensen, Inc., Pawtucket, RI. *ABAQUS Users' Manuals and Theory Manual Version 6.10*, 42
319 edition, 2009.
- 320 [21] A. Hillerborg. The theoretical basis of a method to determine the fracture energy of concrete. *Materials and*
321 *Structures*, 18:291-296, 1985.
- 322 [22] J. Lubliner, J. Oliver, and S. Oller. A plastic-damage model for concrete. *International Journal of Solids and*
323 *Structures*, 25(3):229-326, 1989.
- 324 [23] P. Menetrey. Synthesis of punching failure in reinforced concrete. *Cement & Concrete Composites*, 24:497-507, 2002.
- 325 [24] J. Moe. *Shearing Strength of Reinforced Concrete Slabs and Footings under Concentrated Loads*. Development
326 Department Bulletin D47, Portland Cement Association, 1961.
- 327 [25] R. C. Murthy, G. S. palani, and R. Riyer. State-of-the-art review on fracture analysis of concrete structural compo-
328 nents. *Sadhana*, 34(2):345-367, 2009.
- 329 [26] G. D. Nguyen and A. M. Korsunsky. Development of an approach to constitutive modelling of concrete: isotropic
330 damage coupled with plasticity. *International Journal of Solids and Structures*, 45:5483-5501, 2008.
- 331 [27] J. Pamin and R. de Bort. Simulation of crack spacing using a reinforced concrete model with an internal length
332 parameter. *Archive of Applied Mechanics*, 68:613-625, 1998.
- 333 [28] T. Sain and J.M. Chandra Kishen. Energy-based equivalence between damage and fracture in concrete under fatigue.
334 *Engineering Fracture Mechanics*, 74:2320-2333, 2007.

- 335 [29] A. B. Shuraim and A. I. AL-Negheimish. Influence of wide-shallow-beam flexibility on joist-floor behavior. In
336 *Proceedings of the Third International Conference on Modeling, Simulation and Applied Optimization*, Sharjah,
337 U.A.E, January 20-22 2009. paper 89099.
- 338 [30] A. B. Shuraim and A. I. AL-Negheimish. Design considerations for joist-floors with wide-shallow-beams. *ACI*
339 *Structural Journal*, 108(2):188–196, 2011.
- 340 [31] Ahmed Shuraim. Three-dimensional non-linear modelling aspects of a full-scale reinforced concrete banded-joist floor.
341 In *Proceedings of the Eighth International Conference on Computational Structures Technology*, 2006. paper 13.
- 342 [32] Ahmed B. Shuraim. Efficacy of CFRP configurations for shear of RC beams. *Structural Engineering and Mechanics*,
343 39(3):361–382, 2011.
- 344 [33] F. J. Vecchio. Analysis of shear-critical reinforced concrete beams. *ACI Structural Journal*, 97(1):102–110, 2000.
- 345 [34] F. J. Vecchio. Non-linear finite element analysis of reinforced concrete: At the crossroads? *Structural Concrete*,
346 2(4):201–212, 2001.
- 347 [35] F. J. Vecchio, E. C. Bentz, and M. P. Collins. Tools for forensic analysis of concrete structures. *Computers and*
348 *Concrete*, 1(1):1–14, 2004.

