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Experimental investigation on the dynamic response of RC flat slabs after a sudden column loss

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Abstract

To prevent disproportionate collapse under an extreme loading event, a sudden column loss scenario is often used to ensure the structure has suitable robustness. This study aims to investigate experimentally the dynamic response of reinforced concrete flat slabs after a sudden column loss. Seven 1/3 scale reinforced concrete flat slabs were tested under static load increases or dynamic column removal cases with different supports removed. Reaction forces and deflections were recorded throughout, along with reinforcement strains and concrete cracking patterns. During dynamic tests, a high speed camera was used to capture the dynamic motion. The experiments demonstrated that flat slabs, in general, are able to redistribute their loading effectively after a column loss. Although large levels of damage were observed, collapse due to flexural failure did not occur. However, punching shear was shown to be an issue due to the additional vertical loading on the adjacent supports. The inclusion of continuous bottom reinforcement through a column did not significantly improve the capacity, as the new load path is not primarily through the removed column location. The results also indicate that the dynamic effects due to a sudden column loss can be significant as deflections of up to 1.5 times the static case were measured within the elastic range. It is also shown that

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24 the Dynamic Amplification Factor (DAF) reduces when nonlinear damaging ef-
25 fects are included, which implies conventional code-based design methods for flat
26 slab structures may be over conservative. Additionally, the increase in material
27 strength due the strain rates is not viewed to be significant.

28 *Keywords:* Progressive Collapse, Column Loss, RC Flat Slab, Punching Shear,
29 Dynamic Amplification

30 **1. Introduction**

31 The issue of protecting structures against progressive failure has been a key
32 part of design considerations since the collapse of Ronan Point tower block in
33 1968, where a relatively small gas explosion on the 18th floor led to the collapse
34 of a corner of the structure resulting in several deaths [1]. The issue arose again
35 after the bombing of a federal building in Oklahoma, involving a car bomb dam-
36 aging an external column that was supporting a transfer beam. The partial collapse
37 killed 168 people, the majority due to the collapse progressing through the struc-
38 ture, rather than in the direct blast area [2, 3]. Progressive failure, mainly due
39 to punching shear, has also occurred on flat slab structures, including the Pipers
40 Row car park (1997) [4] and Sampoorn Department store (1995) [5]. As a result
41 of these events, design codes usually require consideration of the potential for
42 progressive collapse. This is commonly achieved by ensuring that the structure
43 can survive the accidental removal of an individual member without experiencing
44 disproportionate damage [6].

45 This situation has been investigated by a number of authors to determine the
46 failure mechanisms and ultimate capacity of Reinforced Concrete (RC) frames
47 after a column loss event. These have included experimental tests on Reinforced

48 Concrete (RC) frames [7–9]. Sasani et al. has conducted a number of tests on real
49 structures to investigate the global response and potential for progressive failure
50 of RC structures [10–13]. The importance of nonlinear effects, both material and
51 geometric, has been highlighted in providing additional capacity and preventing
52 progressive failures. However, sufficient ductility is required to allow yielding and
53 the development of catenary action, otherwise brittle failures may occur.

54 Further consideration has been given to the presence of slab elements, which
55 have been shown to increase the capacity of a structure after a column loss [14,
56 15]. RC slabs have complicated behaviour at high deflections due to their two
57 dimensional nature allowing formation of tensile and compressive membranes
58 [16–19]. Furthermore, their susceptibility to brittle mechanisms such as punching
59 shear [20–23] may potentially lead to progressive collapse.

60 The general behaviour of reinforced concrete slab elements is well known,
61 however, there has been only limited investigations into their performance against
62 progressive collapse. Hawkins and Mitchell [24], Mitchell and Cook [25] and
63 Yagob et al. [26] have addressed some of the issues and Yi et al. [27] recently
64 conducted limited tests to study the quasi-static response. Their results provide
65 valuable insights into the nonlinear behaviour but further tests are still required.

66 Progressive collapse is also a dynamic issue and suitable account needs to be
67 taken of the inertial effects involved after a sudden damaging event [28, 29]. This
68 can be done by conducting a full dynamic analysis of the structure; however this
69 is time consuming and requires detailed information in order to achieve accurate
70 results. Alternatively, an equivalent static case can be considered with a Dynamic
71 Amplification Factor (DAF) applied to the loading. Current design recommenda-
72 tions usually suggest a DAF of 2.0 [30]. The suitability of this value has been

73 studied for some structural types, with some authors suggesting it is over conser-
74 vative [31–34]. However, further investigation is needed for flat slab structures.

75 This study aims to provide much needed experimental evidence for the be-
76 haviour of flat slabs after a sudden column loss, especially considering the nonlin-
77 ear and dynamic effects. Scaled slab models were investigated to simulate the dy-
78 namic response of flat slab elements in different column loss scenarios. Dynamic
79 results under different levels of loading were compared to static tests to assess the
80 additional damage sustained due to inertial effects. Although it is recognised that
81 the simplifications involved in the experimental programme do not completely
82 replicate real structures, the set up is better suited for future modelling with finite
83 element software. Additionally, the key aspects involved can still be considered,
84 allowing future work to focus on the important factors. In particular the details
85 regarding the redistribution of forces and the damage patterns after an extreme
86 event provide an indication of the potential for collapse of a structure. The results
87 from this work will later be used to validate further numerical investigations into
88 this issue.

89 **2. Experimental Procedure**

90 To investigate the behaviour of in-situ RC flat slab structures, seven 1/3 scale
91 simplified substructures were constructed, as shown in Figure 1. These allowed
92 simulation of the removal of a corner, penultimate edge or an internal edge col-
93 umn. Additionally, different reinforcement layouts were considered. Two types of
94 tests were conducted, an increase in static loading and a sudden dynamic column
95 removal. Under the static case, the slab was placed on the supports and the col-
96 umn position under investigation was removed. A uniform load was then imposed

97 across the entire sample by means of sand and gravel bags. Support reactions,
98 deflections, strains and cracking patterns were recorded throughout. Under the
99 dynamic removal, a similarly designed slab cast at the same time as the static, was
100 loaded whilst fully supported. Once the required UDL was achieved, the chosen
101 support was removed and the system allowed to deform and either reach a new
102 equilibrium or experience total failure. During the test the response was recorded
103 with load cells, strain gauges, LVDTs and a high speed camera.

104 **3. Test setup**

105 *3.1. Comparison to real structures*

106 The substructures constructed were based on a typical structural arrangement
107 but with simplifications which made it easier to conduct the experiments and to
108 compare the results to numerical models. Rather than replicate the partial rota-
109 tional and translational restraint that slab-column connections provide, simplified
110 supports were used. Although it is known that this does not represent conditions
111 found in real structures, these assumptions are easier to model for later numerical
112 analysis in order to validate material and mechanical behaviours. Additionally,
113 under a penultimate edge column removal condition, the immediate section of
114 slab is laterally unrestrained, as described by Dat and Hai [18], and therefore the
115 use of such supports is not considered to affect significantly the ability of the slab
116 to form mechanisms such as compressive rings for membrane action.

117 The scaled models were designed to have an equivalent demand to capacity
118 ratio as a full size prototype based on the method presented by Kai and Li [32].
119 For third scale tests, span and depth values were reduced by factor of 3, UDL by
120 1 and reinforcement areas by 3 and therefore the reinforcement ratios were kept

121 constant. The span to depth ratio of all the specimens was 25, which is within the
122 range of values for typical flat slab structures.

123 3.2. Slab design

124 All the slabs were designed to Eurocode 2 [35]. Along with the slab self-
125 weight, the design considered additional finishes of 1kN/m^2 and a variable action
126 of 2.5kN/m^2 . The characteristic cube strength used in the design was 30MPa. De-
127 tails of the test specimens are given in Table 1 along with concrete cube strengths
128 taken from samples on the day of testing. Some variation in the concrete strength
129 can be seen between the tests, and the concrete is below its target strength for tests
130 C-S and C-D. However, for the majority of the discussion, comparisons are only
131 made between tests with similar strengths.

132 For the first six tests, a 2x1 bay subsection of a flat slab structure was con-
133 structed. The specimens were 4100mm x 2100mm in plan with a thickness of
134 80mm. Each sample included two A142 meshes providing 6mm bars at 200mm
135 spacing for both top and bottom reinforcement ($\rho = 0.18\%$). Additional 6mm bars
136 were added over internal supports in the column strip to meet requirements for the
137 hogging moment ($\rho = 0.21\%$). No shear reinforcement was included as the con-
138 crete alone provided enough capacity for a fully supported condition according
139 to Eurocode requirements. This set up was used to replicate both the corner (C)
140 and the penultimate (P) column loss as shown in Figure 1(a). The reinforcement
141 design provided the minimum reinforcement area for both top and bottom steel
142 according to Eurocode 2 [35], a condition that governed in the middle strips. For
143 comparison purposes an equivalent series of tests were conducted without bottom
144 rebar through the column location for the penultimate location tests, designated
145 PC and PR, where C indicates continuous and R indicates reduced.

146 The final test considered a middle (M) column removal using a 4x1 bay sys-
147 tem, constructed in the same manner, with a total length of 8100mm (Figure 1(b)).

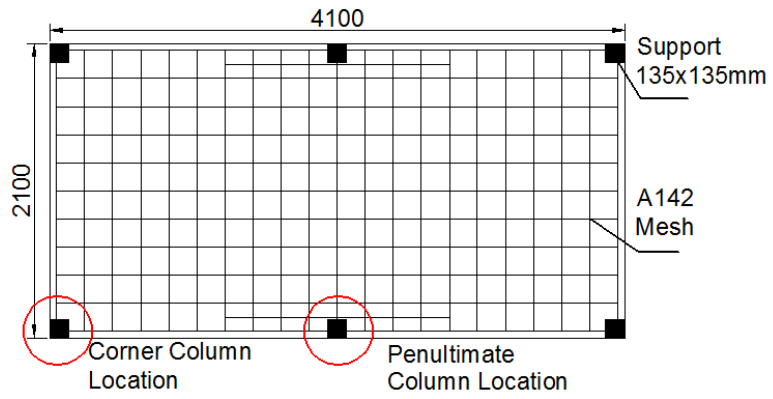
148 *3.3. Support details*

149 The supports were 135mm square steel plates, 25mm thick, on hemispherical
150 bearings to allow rotations. The supports were considered to be pinned, free to ro-
151 tate in any direction and allow the slab to uplift, see Figure 1(c). For the dynamic
152 tests, a temporary support that could be quickly removed was constructed. The
153 temporary support was designed based on a vertical steel bar between two steel
154 plates. The bottom plate rested on a load cell and steel rollers to allow the sup-
155 port to move easily. The removal process used is shown in Figure 2. During the
156 pre-loading period, chocks were placed to prevent lateral movement and a clamp
157 placed around the bar to ensure it remained upright; see Figure 2(a) for details.
158 Once the required loading was reached, and the laboratory area around the test
159 had been cleared, the temporary supports and clamps were removed to create an
160 unstable condition (Figure 2(b)). Finally, a rope attached to the bar was pulled
161 sharply, causing the support base to move and the bar to disengage with the slab,
162 as in Figure 2(c). This system did not cause a true instantaneous removal, how-
163 ever, as the purpose of these tests is to provide information to validate a numerical
164 model, this limitation will be addressed in later work.

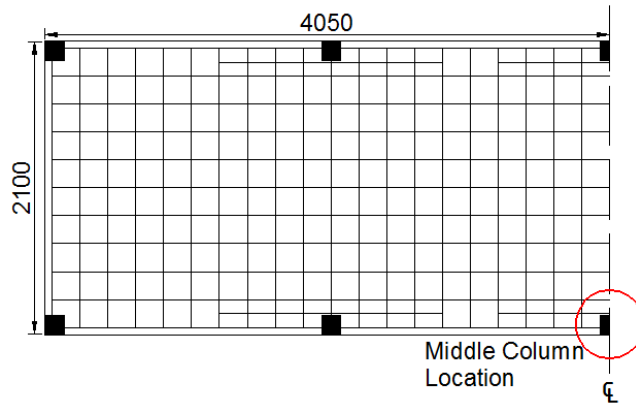
165 An example of the test set up is given in Figure 3 showing a fully loaded
166 sample prior to the sudden removal of the front middle support.

167 *3.4. Instrumentation*

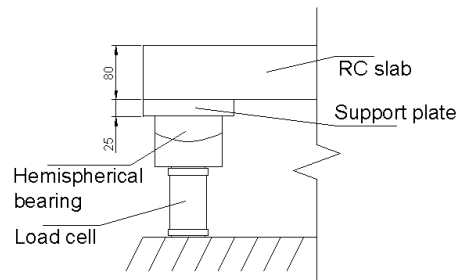
168 Each support included a load cell to measure the vertical reactions, see Figure
169 1(c). The calibration was checked before each test, with a typical uncertainty of



(a) Corner and penultimate removal conditions

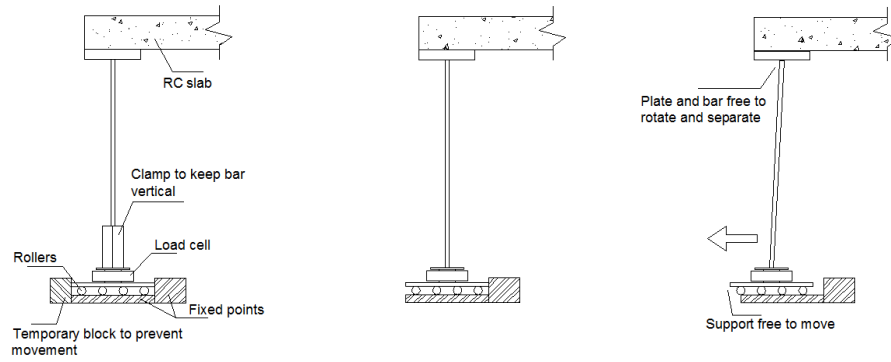


(b) Middle removal condition



(c) Support details

Figure 1: Details of specimens



(a) Fully supported (b) Unstable condition (c) Support being removed

Figure 2: Diagram showing the process of removing the temporary support for dynamic tests

Table 1: Test details and IDs

| Slab ID | Removal Position | Reinforcement | Test Type | Cube Strength (MPa) |
|---------|------------------|---------------|-----------|---------------------|
| C-S | Corner | | Static | 24.4 |
| C-D | Corner | | Dynamic | 26.7 |
| PC-S | Penultimate | Continuous | Static | 33.9 |
| PC-D | Penultimate | Continuous | Dynamic | 37.1 |
| PR-S | Penultimate | Reduced | Static | 33.8 |
| PR-D | Penultimate | Reduced | Dynamic | 35.2 |
| M-D | Middle | | Dynamic | 30.6 |

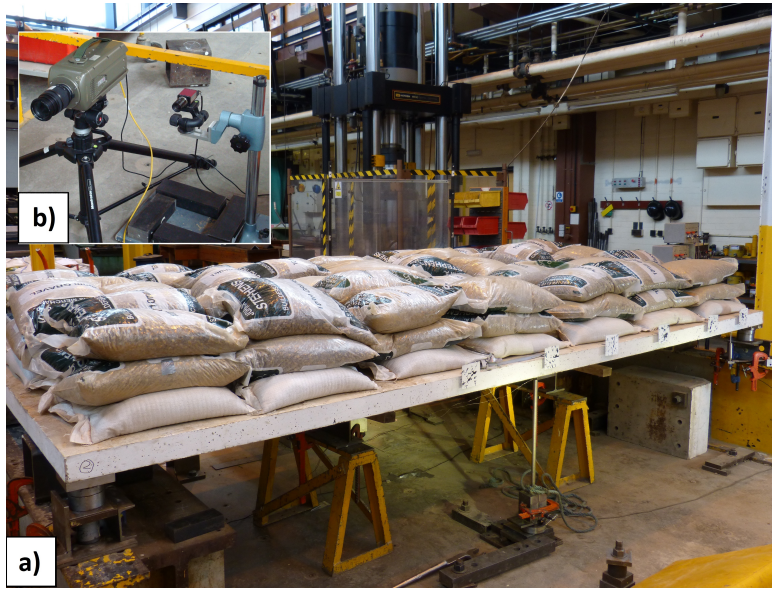


Figure 3: a) Photograph of slab PC-D before dynamic testing; b) Cameras for visual monitoring

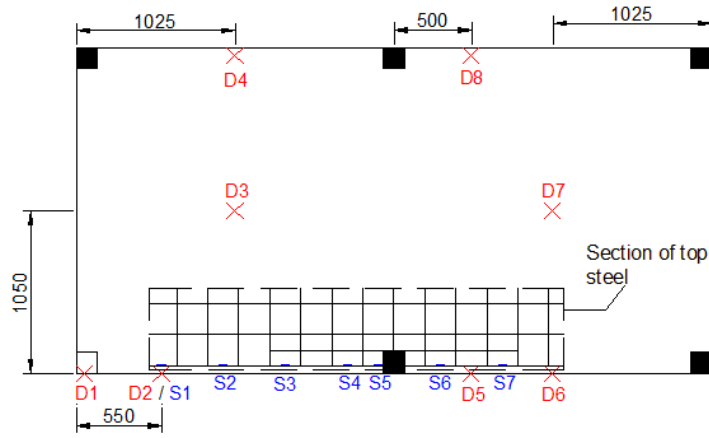
170 50N per load cell, leading to total uncertainties of 0.25, 0.30, 0.45 and 0.5kN for
171 configurations using 5, 6, 9 and 10 load cells respectively. Measuring support
172 reactions before column removal allowed the slab to be balanced correctly. Once
173 a column had been removed, the changes in reactions at the remaining supports
174 allowed the redistribution of forces to be determined. Measurements taken during
175 the tests showed changes in demand to each support as the specimen experienced
176 damage.

177 An array of Linear Variable Displacement Transducers (LVDTs), sampled at
178 250Hz, were placed under each specimen to measure vertical deflections. Around
179 the column loss location, Digital Image Correlation (DIC) techniques were also
180 used to monitor deformations. Camera footage combined with video gauge soft-
181 ware [36] measured the static deflections at points across the sample. The posi-

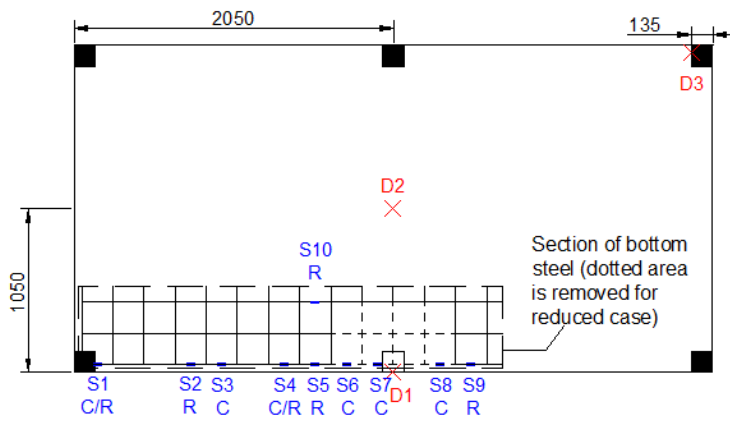
182 tions of the presented measurement points for tests C, PC and PR are given in
183 Figure 4. For the dynamic tests, a Phantom v12.1 high speed camera was utilised
184 to capture the behaviour in the short time period during and after the column
185 removal (see Figure 3(b)). Images were recorded at 2500fps with an exposure
186 of $300\mu s$ and then processed by the video gauge software to obtain deflection
187 readings and to estimate the column removal time. Based on the size of the vi-
188 sual targets, distance of the camera and processing software used, an accuracy of
189 $\pm 0.1mm$ was achieved. The high speed footage was also used to identify crack
190 propagation patterns. Throughout each test, strain gauges were used to determine
191 the stress distributions with the aim of providing information on the critical areas
192 and potential for failure. The locations of the strain gauges on the reinforcing steel
193 that gave usable data are also shown in Figure 4.

194 **4. Experimental Results**

195 For the dynamic removal tests, the high speed footage was analysed to esti-
196 mate the time taken for the support to be removed. This was based on the period
197 between the rope attached to the support becoming taught with the bar starting to
198 move and the moment that either the bar was clearly disengaged with the support
199 plate, or the plate was no longer in contact with the slab. This method is likely to
200 overestimate the removal time, as it does not take account of the condition where
201 the support plate and slab remain in contact, moving vertically at the same rate
202 whilst not transferring forces. A summary of removal times for each test is given
203 in Table 2.



(a) Corner condition

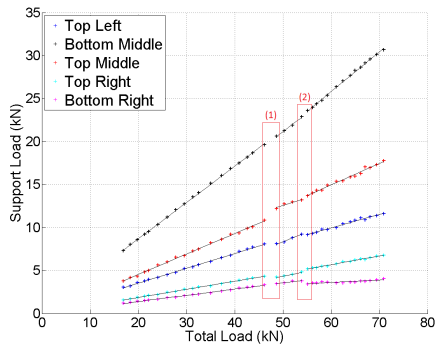


(b) Penultimate removal conditions - Continuous (C) and Reduced (R)

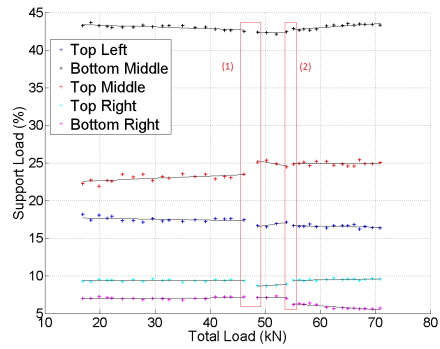
Figure 4: Locations of LVDTs and visual targets (D) and strain gauges (S)

Table 2: Estimated column removal times for each test

| Slab ID | Loading Level (kN/m ²) | Estimated time (ms) |
|---------|------------------------------------|---------------------|
| C-D | 3.0 | 53.2 |
| | 6.8 | 57.0 |
| | 7.7 | 50 |
| PC-D | 2.5 | 40 |
| | 5.6 | 50 |
| PR-D | 2.3 | 52.5 |
| | 5.7 | 39 |
| M-D | 3.1 | 33.7 |
| | 6.7 | 49.2 |
| | 8.5 | 42.6 |
| Average | | 46.7 |



(a) Vertical reaction to each support



(b) Percent of total load to each support

Figure 5: Distribution of forces to supports - Test C-S

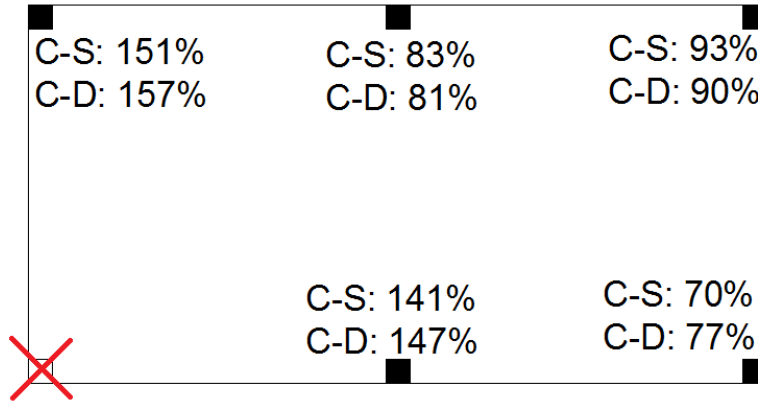


Figure 6: Mean change in distribution of forces to each support after corner column loss - Tests C-S and C-D

204 *4.1. Corner position*

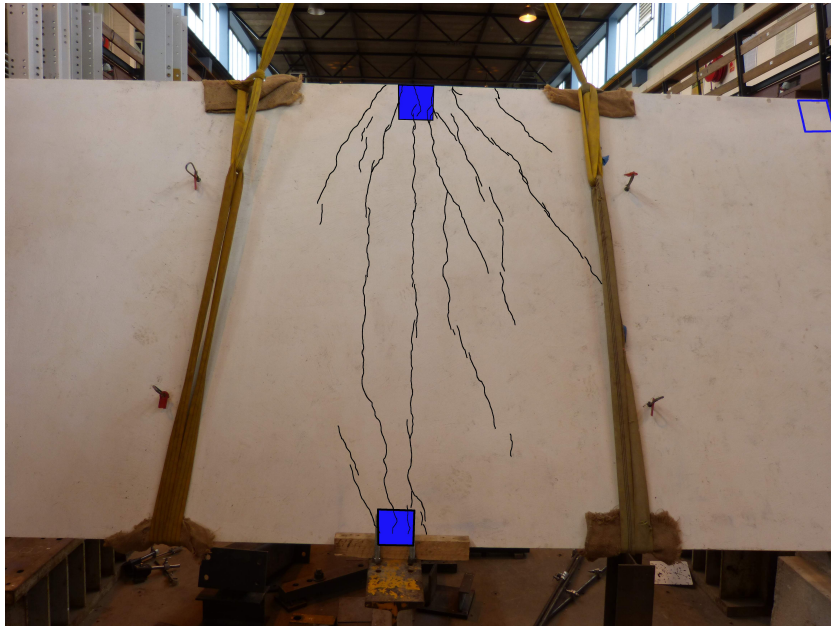
205 *4.1.1. Static loading test*

206 Figure 5(a) shows the vertical reactions at the supports during the corner static
 207 test (C-S) (see Table 1). As it was expected, the reaction forces increased linearly
 208 by increasing the total load in the elastic range. However, beyond 46.2kN total
 209 load (5.4kN/m²) there was a change in distribution (label 1) until approximately
 210 55kN (6.4kN/m²), coinciding with the formation of cracking across the element.
 211 Past this phase (label 2) there is a linear increase of reactions again, though with
 212 a larger deviation from the trend line. The relative distribution of forces to each
 213 support given in Figure 5(b), suggests that the relative demand stayed constant in
 214 the elastic and final ranges. Between labels 1 and 2 there was again a noticeable
 215 change as redistribution of forces occurred due to extensive crack formation. Con-
 216 sidering, however, the entire range, suggests that a linear model could be used to
 217 describe the relationship.

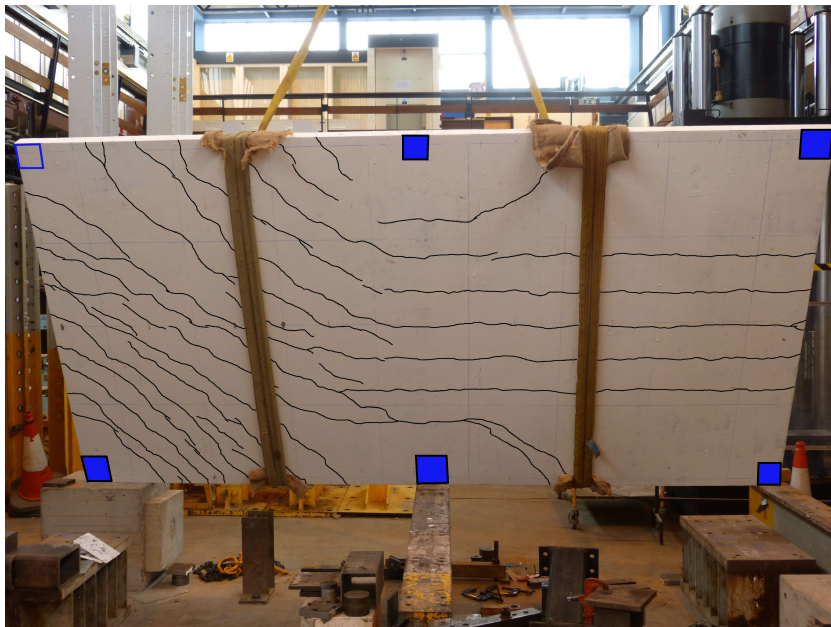
218 Comparison between the averaged reaction forces for fully supported and
219 damaged conditions (see Figure 6) indicates that the two orthogonally adjacent
220 supports experienced a 41-57% increase in their vertical reaction while all other
221 supports had a decrease in demand. It should be noted that C-S and C-D showed
222 similar ratios, indicating dynamic removal did not change the final distribution of
223 reaction forces.

224 At higher levels of loading, significant flexural cracks formed due to the large
225 increase in hogging moments in both tests, initially on the top surface over the ad-
226 jacent support (Figure 7(a)). Sagging flexural cracks also formed on the underside
227 as the slab now spanned diagonally between the two supports nearest the removal
228 location (Figure 7(b)). The location of permanent supports (solid boxes) and the
229 removed support (outline) are annotated in this figure.

230 The plot of normalised displacements, deflection (δ) / slab thickness (t), against
231 load in the damaged bay area (Figure 8(a)) shows an initial linear response. How-
232 ever, after a load of 4.6kN/m² flexural cracks start to form resulting in a decrease in
233 stiffness to around 57% of the initial value. At 6.0kN/m², when the peak displace-
234 ment equals 0.19 times the slab depth, there is a discontinuity due to significant
235 cracking over the adjacent support along with yielding of the reinforcement. This
236 led to an increase in displacements across the entire sample, with the maximum
237 exceeding half the slab depth. After this, there was a brief stiffening phase be-
238 fore a final softening with a relative stiffness of 6% of the elastic range. The slab
239 continued to carry additional load until the test was aborted at 8.2kN/m². In the
240 adjacent bay, shown in Figure 8(b), once damage occurred there was a jump in re-
241 sponse observed in the middle (point D7, Figure 4(a)) due to the flexural sagging
242 cracks in that area. The high deflections in the damaged area also led to a relative

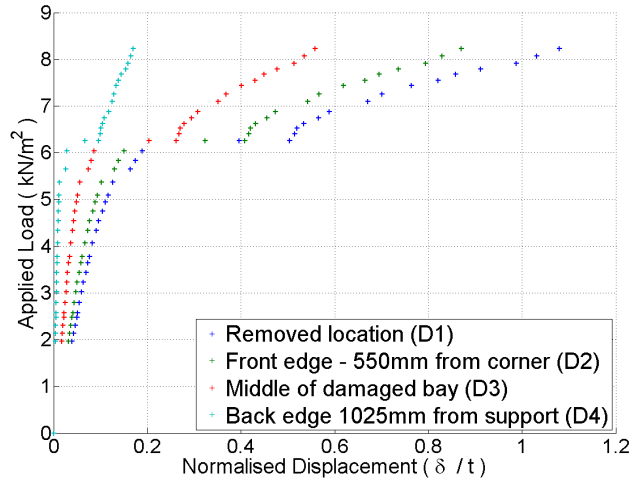


(a) Top surface

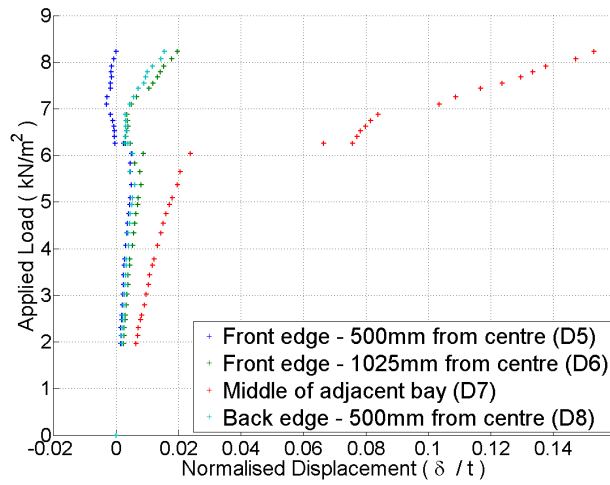


(b) Bottom surface

Figure 7: Annotated flexural cracks after corner column loss



(a) Displacements in the damaged bay



(b) Displacements in the adjacent bay

Figure 8: Load against normalised displacements - Test C-S

243 uplift due to the large rotation around the central support (point D5). The discon-
244 tinuous response corresponds to the changes in reaction forces seen in Figure 5,
245 as discussed in the previously.

246 The strain data in Figure 9 provides a further understanding of the damage
247 profile. Strain readings have been corrected against the fully loaded condition
248 under the slab's own self weight, i.e. the results demonstrate the change from
249 the starting condition, and then normalised against the yield strain. Below a load
250 of 4.1kN/m^2 , strains on the steel over the central column were relatively low.
251 However, after the formation of flexural cracking there was a peak in strain on the
252 damaged side of the support at 6.04kN/m^2 , corresponding to the large increase in
253 displacements seen in Figure 8(a). As loading increased there was local yielding
254 of the reinforcement in this area, while other areas remain well below the yield
255 strain.

256 4.1.2. *Dynamic removal test*

257 The normalised displacements (displacement/ slab thickness) for dynamic re-
258 moval at three different levels of loading are plotted in Figure 10 for the removal
259 location and the middle of the adjacent bay (Points D1 and D7 in Figure 4(a)).
260 Peak displacements, damped natural frequency and damping ratio results for these
261 tests are compared in Table 3.

262 At 3.0kN/m^2 the structure was within the elastic range resulting in small de-
263 flections (7% and 5% of slab depth for peak and final displacements respectively).
264 The low total mass resulted in a high frequency response, and as no damage oc-
265 curred there was little dissipation of the energy. The low damping ratio ($\zeta = 0.01$)
266 caused the system to take several seconds to return to its static equilibrium posi-
267 tion.

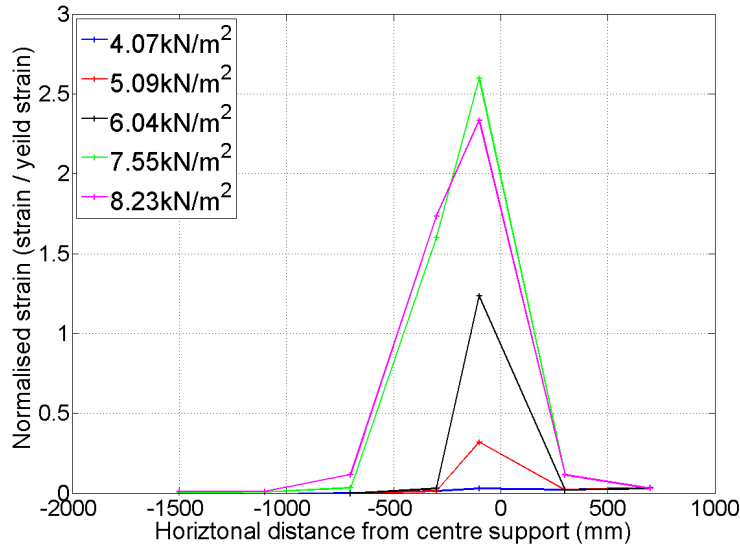


Figure 9: Normalised strain against position for top reinforcement bars - Test C-S

268 The specimen was then reset to the starting position and the static load in-
 269 creased to 6.8kN/m^2 , just within the plastic region from the earlier static condi-
 270 tion. Much higher deflections, peaking at almost 60% of the slab depth, were
 271 measured. Thin hogging cracks were observed, which resulted in a higher energy
 272 dissipation and a larger damping ratio ($\zeta = 0.24$), however overall damage was
 273 not extensive.

274 For the final case the load was increased to 7.7kN/m^2 and the test repeated.
 275 Figure 11 shows the power density spectrum from a Fourier transform of dis-
 276 placement readings following a corner column loss at different load levels. The
 277 results indicate that for the slab in the plastic region (i.e. 7.7kN/m^2), the large de-
 278 flections and resulting damage created a different response to the single dominant
 279 frequency peaks seen before. As cracks had already formed during the 6.8kN/m^2

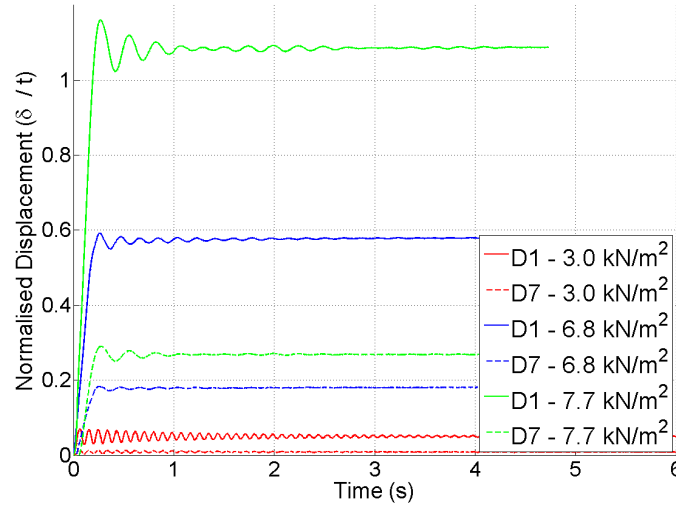


Figure 10: Normalised displacement against time after column removal at different positions and loading - Test C-D

280 test and subsequently widened in the next case, the friction at the crack face was
 281 reduced resulting in the smaller damping ratio observed at 7.7kN/m² (Table 3).
 282 Additionally, the pre-existing damage may have been a factor for the two fre-
 283 quency response seen. At this loading, peak deflections exceeded 110% of the
 284 slab depth but did not lead to complete failure.

285 Within the elastic range, the amplitude between the first peak and first dip is
 286 60.7% of the maximum displacement, indicating the structure returns relatively
 287 close to its initial state. Once permanent damage had occurred both these ratios
 288 drop considerably as seen in Table 3.

289 The strain data collected during a dynamic removal also allowed estimation
 290 of the strain rates, $\dot{\epsilon}(t)$. The tensile strain rates against time for the top steel are
 291 presented in Figure 12 for the three loading levels. Each line shows the maximum

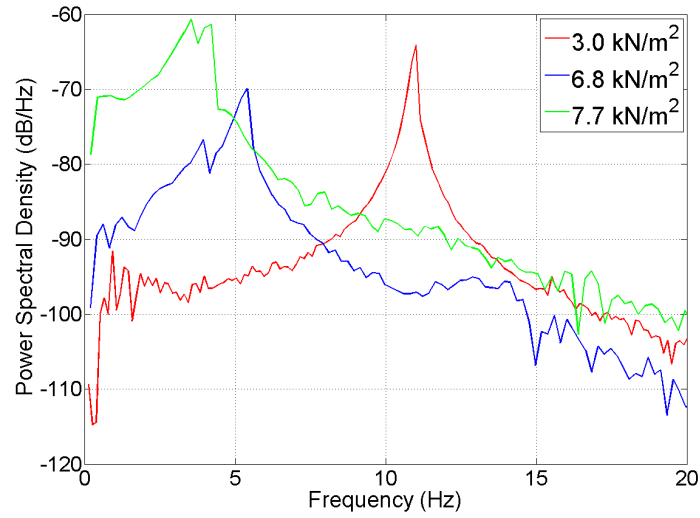


Figure 11: Power spectral density of displacement following corner column loss at different load levels - Test C-D

| Loading (kN/m ²) | 3.0 | 6.8 | 7.7 |
|-------------------------------|------|------|-----------|
| Normalised Peak | 0.07 | 0.59 | 1.16 |
| Amplitude / Peak (%) | 60.7 | 7.36 | 11.91 |
| Peak / Final Displacement | 1.42 | 1.02 | 1.07 |
| Damped Natural Frequency (Hz) | 11.0 | 5.41 | 3.54/4.21 |
| Damping Ratio | 0.01 | 0.24 | 0.123 |

292 strain rate that occurred at any monitored position, at each time step.

293 Strain gauge S5, positioned next to the central support, see Figure 4(a), ex-
294 perience a much higher strain rate for each loading level. Since the graph only
295 presents the maximum value, the response of the other gauges is hidden. There-
296 fore, a second line is plotted excluding this sensor. Additionally, the key strain
297 data with time, adjusted against the strain readings at the fully support condition,
298 is also plotted on the second vertical axis to allow further comparisons.

299 In Figure 12(a), the elastic case, most the sensors on the top steel show very
300 low levels of strain rates, with only sensor S5 showing a strong peak. However, it
301 is clear that the peak strain occurs a period of time after the peak strain rate. This
302 is significant in considering the influence of strain rate effects in increasing the
303 material tensile capacity during sudden column losses. At 6.8kN/m^2 of loading,
304 shown in Figure 12(b), a similar pattern is seen, however there is still a reasonable
305 peak at other locations. Overall, high strain rates are observed here with a max-
306 imum rate of 0.153s^{-1} occurring just before the maximum strain. The change in
307 maximum strain from the fully supported case suggests that the steel has yielded
308 in this area; this may explain the localised high strain rate and also affect the final
309 results.

310 The final loading case presented in Figure 12(c) shows a different response.
311 The largest strain rate does again comes from sensor S5 (next to the central sup-
312 port), the rates and change in strains are smaller than the previous case. This is
313 most likely due to the plastic deformations that occurred. Of further interest is
314 sensor S3, see Figure 4(a) for its location. As this position was previously closer
315 to the middle of the span, it was under a compressive condition and then changed
316 to a tensile state due to the column loss. This change demonstrated itself by a

Table 4: Details of shear failures

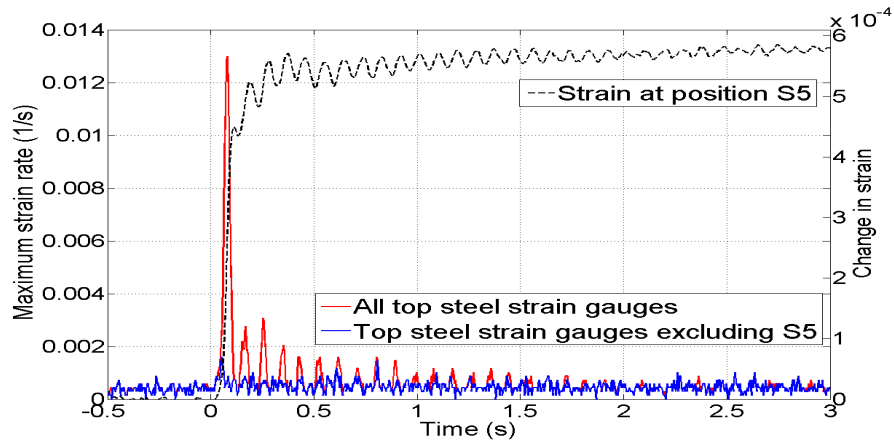
| Slab ID | Max loading (kN/m ²) | Max displacement (δ/t) | Shear failure | Loading type at failure | Initial location | Further failure |
|---------|----------------------------------|---------------------------------|---------------|-------------------------|--------------------|---------------------------------|
| C-S | 8.2 | 1.08 | No | | | |
| C-D | >7.7 | 1.54 | Yes | Static push | Back left corner | Bottom middle down ^a |
| PC-S | 6.4 | 2.23 | No | | | |
| PC-D | 6.8 | 1.71 | Yes | Static ^b | Front left corner | |
| PR-S | 6.7 | 1.67 | Yes | Static | Front left corner | Front right corner |
| PR-D | 5.7 | 2.12 | Yes | Dynamic | Front right corner | Front left corner |
| M-D | 9.2 | 0.74 | No | | | |

^aAfter the final dynamic test a large load was applied over the removed corner to cause complete failure.

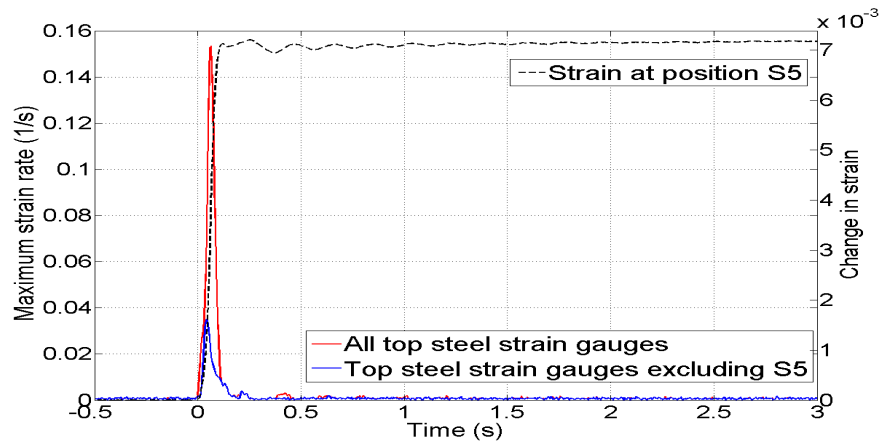
^bAfter the final dynamic test, loading changed to a static UDL.

317 delay in response before the large tensile deformations occurred leading to large
318 permanent strains. The peak rates were 0.031 and 0.034s⁻¹.

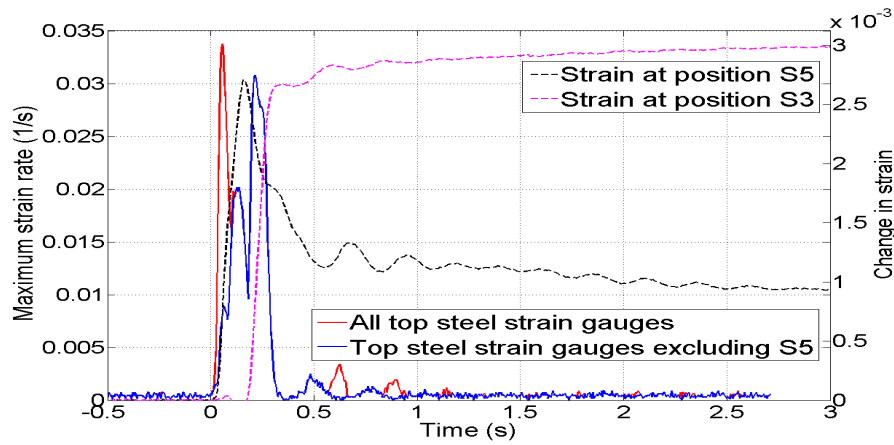
319 Finally the sample was loaded to failure, which occurred due to punching shear
320 at the two adjacent supports as shown in Figure 13. Table 4 gives the shear failure
321 details of all the slabs tested.



(a) 3.0 kN/m² loading



(b) 6.8 kN/m² loading

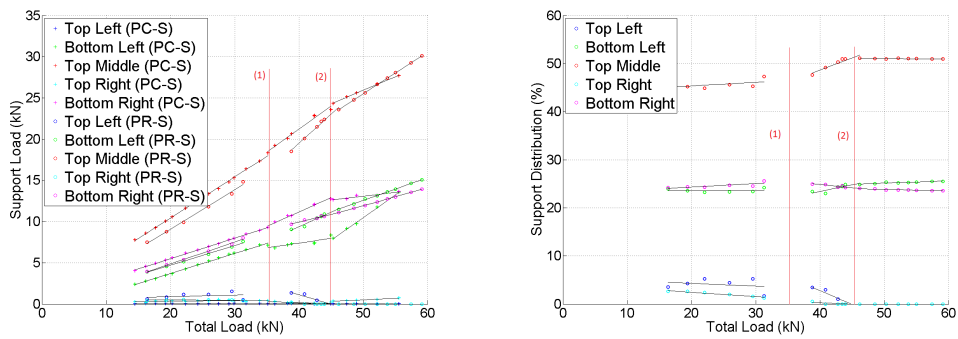


(c) 7.7 kN/m² loading

Figure 12: Maximum steel strain rates against time. Also showing changes in strain against time.



Figure 13: Final state of corner removal case after shear failure - Test C-D



(a) Vertical reaction to each support - Tests PC-S and PR-S (b) Percent of total load to each support - Test PR-S

Figure 14: Distribution of forces to supports - Tests PC-S and PR-S

322 *4.2. Penultimate position*

323 *4.2.1. Static loading test*

324 The load increase to each support for two Penultimate removal cases with
325 static loading are shown in Figure 14(a). Similar responses are observed for the
326 two conditions with nearly all points showing a simple linear relationship at low
327 loading at equivalent rates. The back middle support takes the highest proportion
328 of loading, followed by the front corners.

329 Flexural cracking occurred at 35kN and 30kN of total load, for PC-S and PR-S
330 respectively, which was followed by a period of redistribution of reaction forces
331 across the samples until approximately 45kN, between labels (1) and (2). After
332 this stage the distribution remains reasonably constant until failure.

333 The change in support reaction distribution occurred principally due to uplift at
334 the back two corners, as a result of the large downward deflection in the middle.
335 What little load those supports had been carrying was then taken by the other
336 supports (Figure 14(b)), primarily the back middle.

337 The bottom left location in PC-S shows a more dramatic change. This was due
338 to the load cell rotating at higher deflections, an issue that was corrected for in
339 other tests by stabilising the load cell horizontally, and does not indicate a change
340 in loading on the support.

341 Deflections of PC-S and PR-S are given in Figure 15 for the positions identi-
342 fied in Figure 4(b). It is shown that there is a clear linear response across all parts
343 of the slab before cracking occurs. Additionally, the initial stiffness of the two re-
344 inforcement cases was identical. Both cases started to crack at similar points, with
345 a slight reduction in stiffness observed after 3.4kN/m². This corresponds to a peak
346 normalised displacements of 0.1. However, after peak displacement of 0.13 times

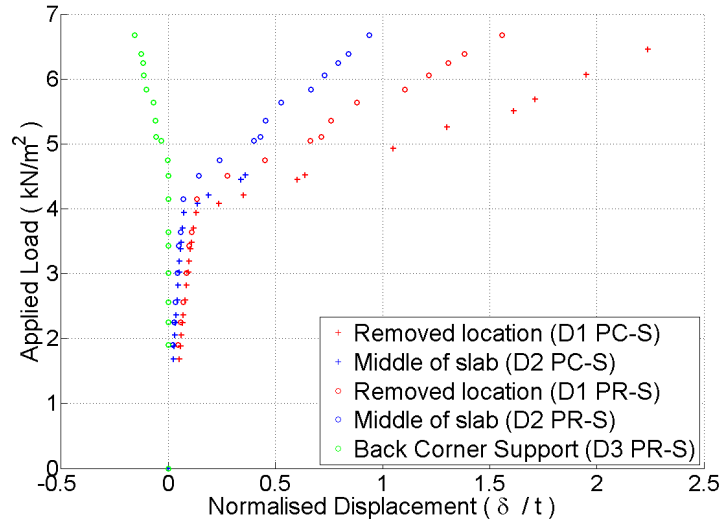
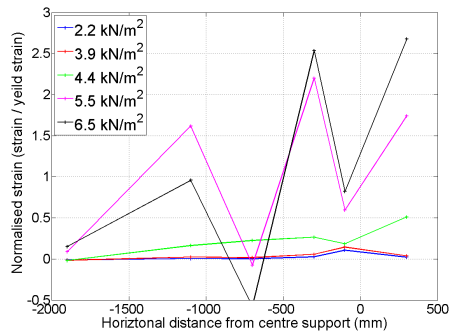


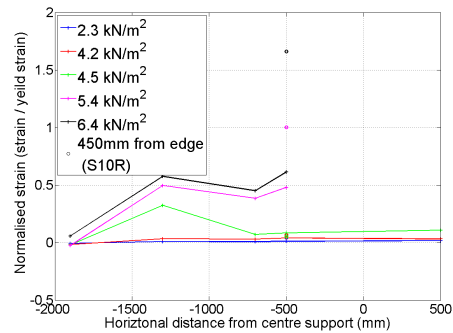
Figure 15: Load against normalised displacement for PC-S and PR-S

347 the depth there is a significant reduction in stiffness due to more extensive flexural
 348 cracking. After this point there was a linear trend for the remaining data, though
 349 the new stiffnesses were less than 5% of the initial values. In the corner removal
 350 case there was a large increase in displacements as cracking formed, which does
 351 not occur here. The geometry of these tests meant that sagging cracks were the
 352 most significant form of damage and these were spread out across the midspans
 353 and so did not cause the sudden drop in stiffness observed from the very localised
 354 hogging cracks in the previous test. The results also demonstrate the uplift ef-
 355 fect experienced at the back support (point D3 in Figure 4(b)), as shown by the
 356 negative displacement.

357 The reduced case experienced a sudden shear crack of the front left support
 358 at 6.7kN/m^2 with an approximate shear force of 15.1kN . The corner sections had
 359 a designed shear capacity of 12.6kN according to Eurocode 2. As soon as this



(a) Continuous reinforcement - Test PC-S



(b) Reduced reinforcement - Test PR-S

Figure 16: Normalised strain against position for bottom reinforcement bars

360 failure occurred, the second front corner support also failed by shear (see Table
 361 4). Test PC-S was ended due to safety concerns at a lower loading than the level
 362 that caused shear failure in PR-S, although the design shear capacity had already
 363 been exceed. Had the test been continued it is likely that a similar failure would
 364 occur. The rotation of the load cells, and therefore support conditions, for the
 365 continuous reinforcement test also resulted in the higher deflected profile without
 366 causing shear failure.

367 Figure 16 shows the strain profiles of the bottom reinforcement bars for tests
 368 PC-S and PR-S. The location of the strain gauges are shown in Figure 4(b)). For
 369 the continuous reinforcement case (Figure 16(a)), the middle area ($\pm 500\text{mm}$ from
 370 the removed column) had the highest strain for loading less than 4.5kN/m^2 . How-
 371 ever, once cracking started there was a significant change in the stress distribution
 372 and yielding occurred across much of the length of the monitored bar. The drops
 373 in values can be explained by local variation in stress due to the effect of concrete
 374 de-bonding around the steel. Removing the central bottom flexural steel from the



Figure 17: Failure of slab PR-D captured from high speed camera - a) Flexural cracking; b) Shear crack

375 column location ($\pm 400\text{mm}$ from the centre) resulted in a different response (see
376 Figure 16(b)). Note that for loading greater than 4.5kN/m^2 the strain gauge at
377 500mm (gauge S9R) failed and its values have been removed. Due to the non-
378 continuous state of the reinforcement, smaller strain was observed at equivalent
379 loading and positions compared to PC-S, and none of the steel bars yielded. How-
380 ever, an extra gauge (S10R) at -500mm horizontal distance and 450mm away from
381 the edge, is included (marked with o's). This sensor was on the first bar that is
382 continuous along the length and did yield. Strain gauges on the top surface of the
383 concrete, along with visual inspections, revealed that the concrete never under-
384 went crushing.

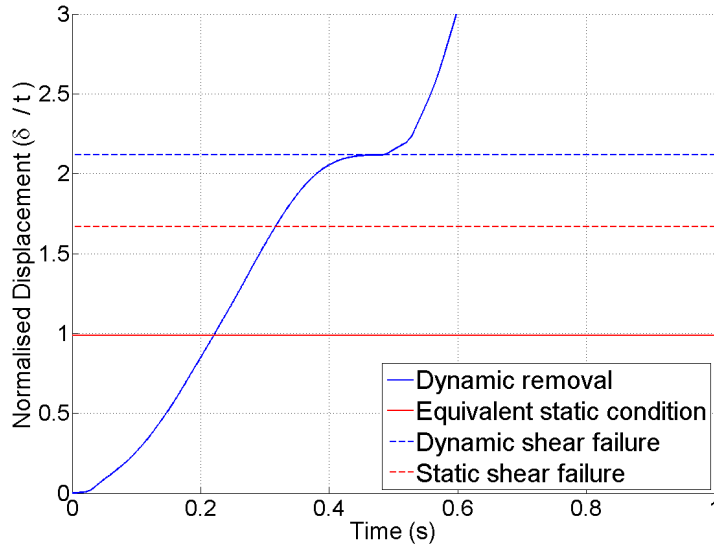


Figure 18: Displacement against time for PR-D at 5.7kN/m²

385 4.2.2. *Dynamic removal test*

386 The failure of the reduced reinforcement condition under dynamic removal
 387 with 5.7kN/m² of loading is shown in Figure 17, captured by the high speed camera.
 388 A wide flexural crack initially occurred due to the lack of tensile reinforcement,
 389 before a final shear crack formed leading to complete shear failure, see
 390 Figure 17(a) and (b) respectively.

391 The normalised deflections against slab thickness for this test are plotted in
 392 Figure 18, along with the static case at equivalent loading to demonstrate the in-
 393 crease in deflections experienced due to the dynamic effects. In the dynamic case
 394 there was a peak displacement of 2.12 times the slab depth, before the shear crack
 395 formed at 0.47 seconds. Comparing the results to the static test gives a dynamic
 396 displacement amplification ratio of 2.14. However, due to the nonlinear relation-

397 ship this value is not useful. Extrapolating beyond values from the static force
398 displacement line (Figure 15) gives an equivalent force DAF of only 1.35, based
399 on the assumption that shear failure does not occur. The reduction in stiffness
400 caused by the initial flexural damage might have caused the much higher deflec-
401 tions observed. Furthermore, the maximum vertical reaction at adjacent supports
402 occurred as the slab reaches a temporary static condition at its maximum deflec-
403 tion, this delayed the shear crack forming and allowed higher deflections to be
404 reached. For further comparisons, details of shear failures are given in Table 4.

405 Considering the strain rate data for the two tests, shown in Figures 19 and 20,
406 demonstrates that moderately high strain rates occurred after the sudden column
407 loss at the higher loadings, in the order of $0.2\text{-}0.3\text{s}^{-1}$. However, as was seen in the
408 corner loss case, the peak strain, and therefore highest stress, in the material occurs
409 after the maximum strain rate. Additionally, at this point, the rate was close to its
410 minimum as the sample was at a temporary rest position between oscillations.
411 Test PC-D in Figure 19 shows that the strain rates in the elastic test are relatively
412 small, around 0.02s^{-1} . Furthermore, most of the monitored points also had small
413 strain rates even at the higher loading. However, strain gauge S3, see Figure 4(b),
414 did show much higher values. This is to be expected from comparing to the static
415 case in Figure 16(a) as that location clearly undergoes yielding. Of further interest
416 is strain gauge S7, which was positioned at the support that was removed. This
417 location quickly switched from a compressive, hogging state, to a sagging, tensile
418 condition, which explains its high strain rate immediately after removal. However,
419 this area became less critical due to further damage occurring across the slab.

420 Figure 20, showing strain rate data from test PR-D, gives a comparison be-
421 tween maximum strain rates and maximum strain. Additionally, the vertical line

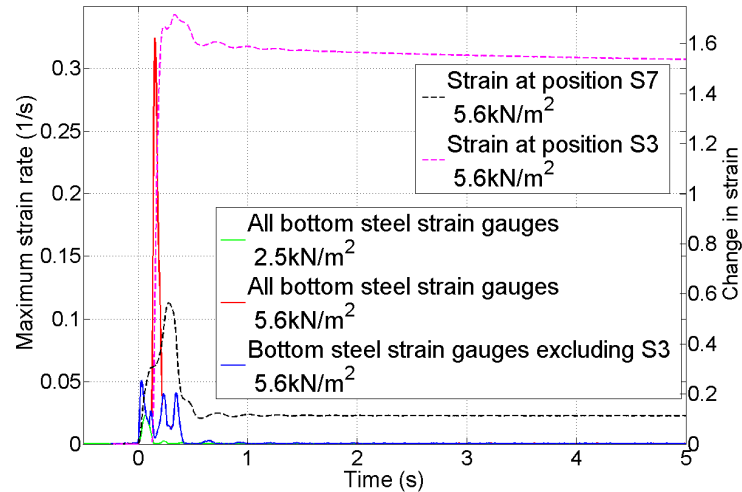


Figure 19: Maximum steel strain rates against time. Also showing changes in strain against time.
 - Test PC-D

422 indicates the time at which the shear crack formed. Considering strain positions
 423 S3 and S7, it can be seen that the sample had reached its maximum deflection and
 424 strain and was about to continue its oscillation when the slab failed due to shear.
 425 At this time the strain rates were very low at all points across the slab.

426 The flexural cracks on the underside of the test specimens are shown in Fig-
 427 ure 21, Permanent supports (solid boxes) and the removed support (outline) are
 428 shown. In both cases there were primary cracks spreading perpendicular to the
 429 new support arrangement. The position of the bottom reinforcement mesh is also
 430 indicated to show that the orthogonal cracks in the middle area follow the steel po-
 431 sitions. This is especially pronounced in PC-D, where the diagonal cracks reach
 432 right to the centre line. Whereas for the reduced case (PR-D) it is shown in Figure
 433 21(b) that the cracks were non-continuous at the column loss location and prop-

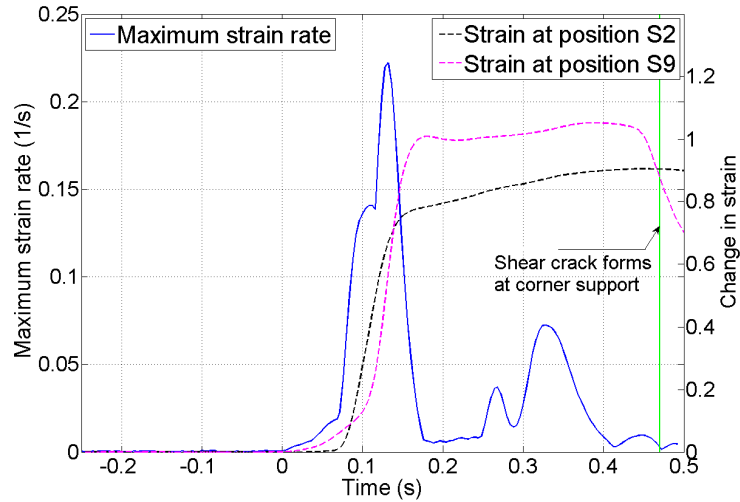
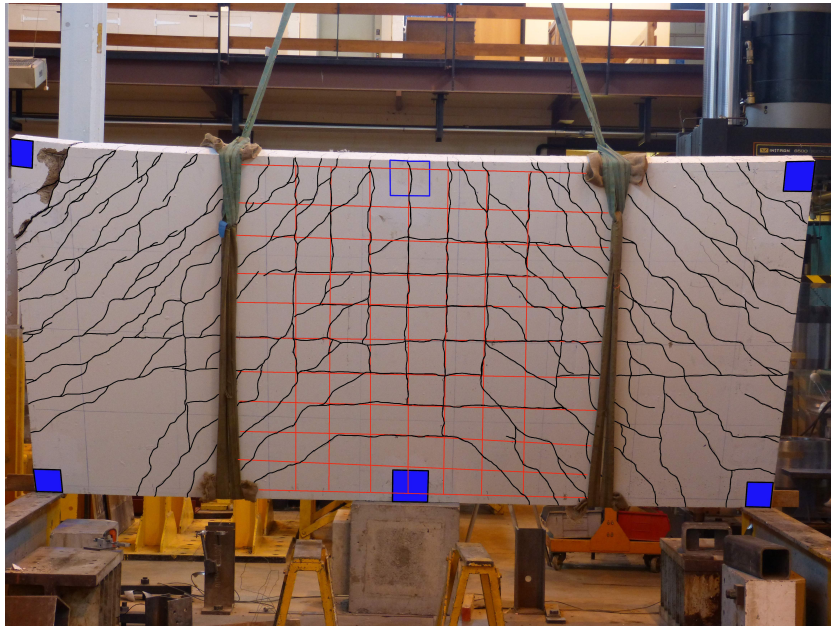


Figure 20: Maximum strain rates against time at 5.7kN/m^2 . Also showing changes in strain against time - Test PR-D

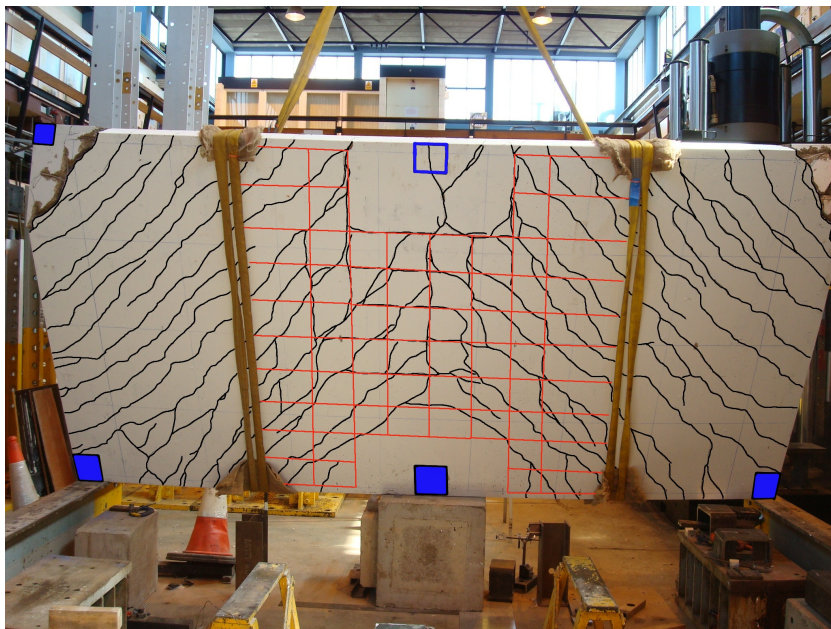
434 agated around the edge of column area, following the reinforcement lines, rather
 435 than exploiting the lack of tensile reinforcement in the central area (c.f. Figure
 436 21(a)). However, these cracks were wider and deeper than at other locations and
 437 in other tests. For all penultimate removal tests, there was only minimal hogging
 438 cracking on the top side running down the centre line, which was followed by
 439 shear failures on one or both of the front corner supports (see Table 4).

440 4.3. Middle position

441 Test M-D was a 4x1 bay continuous slab with a middle column dynamically
 442 removed. The change in support reactions from fully supported to the damaged
 443 case are shown in Figure 22. Similar to the previous tests, the largest increase
 444 in reaction occurred at the supports immediately adjacent to the removal point,
 445 whereas the supports further away have a relative reduction in vertical reaction



(a) Continuous reinforcement - Test PC-D



(b) Reduced reinforcement - Test PR-D

Figure 21: Annotated bottom surface flexural cracks and reinforcement after penultimate column loss

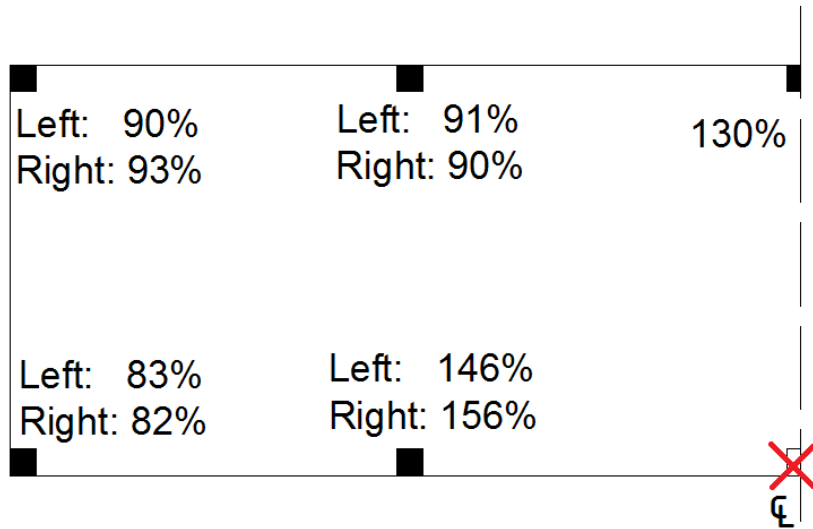


Figure 22: Mean change in distribution of forces to each support after corner column loss - Test M-D

446 force.

447 Dynamic removal tests were conducted at different loadings and normalised
 448 deflections were calculated using images captured from the high speed camera.
 449 Figure 23 compares displacement at the removal location in test M-D for different
 450 load levels. Although this arrangement in general shows the same behaviour as
 451 the previous tests, there was a significant reduction in the normalised displace-
 452 ments. Comparing the key results given in Table 5 with the equivalent loading for
 453 the corner removal case (Table 3), gives a reduction of 55% for the peak displace-
 454 ment in elastic cases. Additionally, at the next loading level (6.8/6.9kN/m²), the
 455 continuous slab peak displacement was only 0.09 times the slab depth, compared
 456 to 0.59 in test C-D. As this set up caused a stiffer structure compared to the corner
 457 removal tests, displacements are expected to be smaller. Furthermore this also

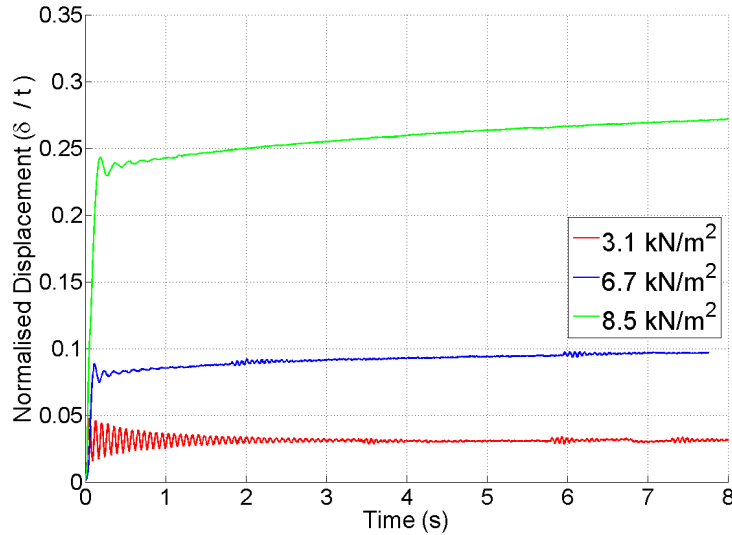


Figure 23: Displacements against time at the removal location for different loadings - Test M-D

458 meant that the damage, and therefore reduction in stiffness, occurred at a higher
 459 level of loading for this case.

460 At lower levels of load, the bays adjacent to the damaged area experienced
 461 a slight uplift, as shown by the negative displacements in Figure 24, due to the
 462 slab rotating inwards towards the removed support. Initially, after the column
 463 was removed in the 8.5kN/m² test, there was a brief uplift (label (A) in Figure
 464 24), however, the damage sustained across the slab resulted in a final downward
 465 motion.

466 Cracking of both the top and bottom surfaces of the concrete led to large plastic
 467 deformations and the drift observed in Figures 23 and 24. However, collapse due
 468 to total flexural failure did not seem likely and shear cracks did not form within
 469 the levels of loads tested (see Table 4).

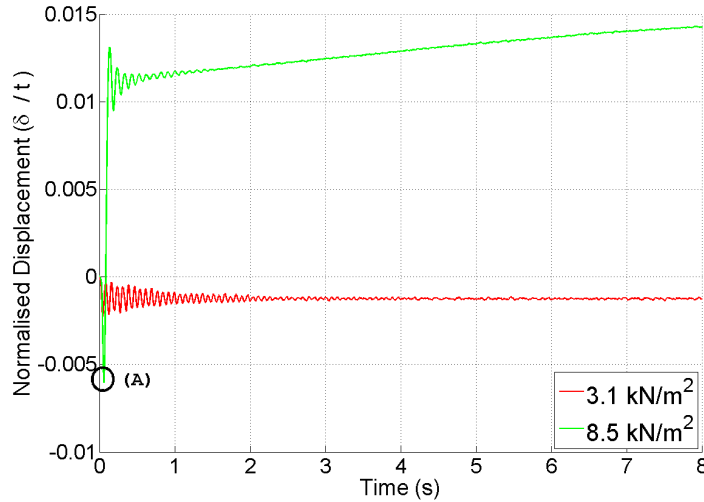


Figure 24: Displacements against time at the center of the adjacent bay for different loadings Test M-D

470 Based on experimental results, Table 5 gives the values of dynamic effects for
 471 three loading levels. In the elastic range (i.e. 3.1 kN/m^2) there was a higher natural
 472 frequency and a smaller damping ratio. For this the influence of inertial effects
 473 on a lightly loaded slab can be seen, resulting in a high peak to final displacement
 474 ratio of 1.54. Again increasing the load decreased the frequency of oscillation
 475 and increased the damping ratio. For the higher load cases there was an initial
 476 dynamic behaviour then, as the major dynamic motion was damped out, the slab
 477 underwent further downward deflections under its own self-weight. These dis-
 478 placements became larger than the initial dynamic peak and resulted in further
 479 deflections as the slab returned to a static condition. This caused the peak to fi-
 480 nal displacement ratios of less than 1 presented in Table 5. This behaviour was
 481 a result of the damage, and therefore reduction in stiffness, sustained during the

Table 5: Results from dynamic removal - Test M-D

| Loading (kN/m ²) | 3.1 | 6.9 | 8.5 |
|-------------------------------|-------|-------|-------|
| Normalised Peak | 0.05 | 0.09 | 0.24 |
| Amplitude / Peak (%) | 67.31 | 15.98 | 5.82 |
| Peak / Final Displacement | 1.54 | <0.92 | <0.90 |
| Damped Natural Frequency (Hz) | 13.4 | 8.55 | 6.00 |
| Damping Ratio | 0.017 | 0.219 | 0.204 |

482 dynamic response. However, after the period of recording the slab came to rest
 483 and complete failure did not occur.

484 Figure 25 shows a photograph of the underside of slab M-D after the test was
 485 completed with the cracks annotated. The primary cracking pattern is shown in
 486 black. In this specimen the two-way spanning nature of a slab structure after a
 487 column loss is clear by the diagonal cracks. The red lines are secondary flexural
 488 cracks that follow the reinforcement lines. As the slab was not continuous in both
 489 directions these cracks were more extensive than would be expected in a typical
 490 structure. The top cracking due to the increased hogging moments over the adja-
 491 cent supports was almost identical to the corner removal case shown previously
 492 in Figure 7. These cracks followed the same pattern as seen in Figure 21, though
 493 were less extensive due to the smaller deflections and the influence of adjacent
 494 bays.

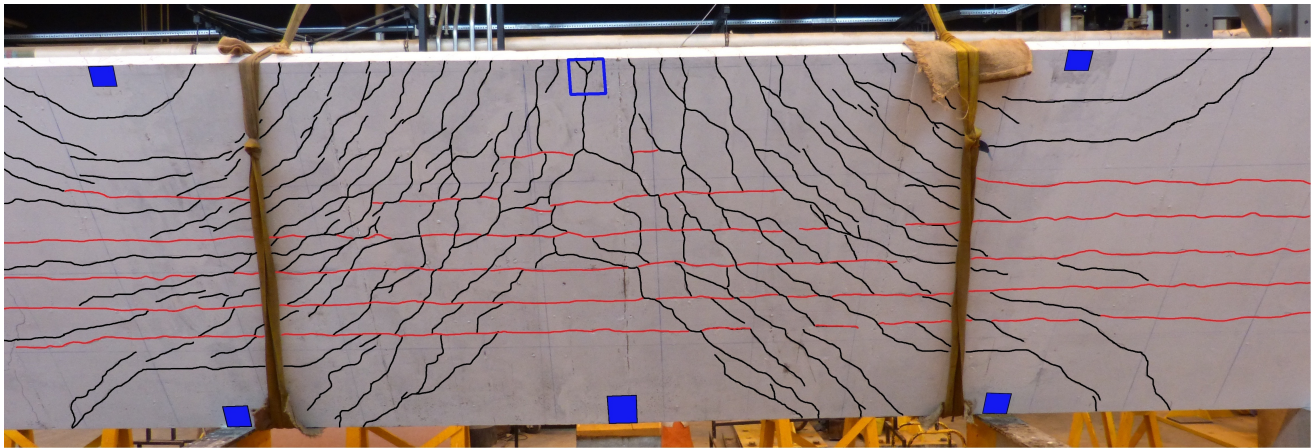


Figure 25: Annotated underside cracking pattern for continuous slab Test M-D

495 **5. Discussion**

496 These tests sought to simulate the effect of a column loss on a flat slab system.
497 The measured reactions forces indicate that each slab was balanced suitably at the
498 start of each test and that the loading was applied evenly across its surface. Anal-
499 ysis of the high speed footage shows that the support was typically completely
500 removed within 50ms. Although this is slower than a true instantaneous column
501 loss scenario caused by an explosion [3, 37], similar removal rates were achieved
502 for all tests allowing comparisons to be made. Furthermore the results still demon-
503 strate the effects of a quick removal. A quicker removal scenario may increase the
504 dynamic effects slightly and will be considered in later numerical analysis.

505 *5.1. Force redistribution*

506 The reaction force distribution and the cracking patterns shown in Figures 6, 7,
507 14, 21, 22 and 25 give a good indication of the change in load paths that a damaged
508 slab experiences. The test observations indicate that the bending profile becomes

509 truly two-dimensional, with new spans primarily acting diagonally between the
510 nearest supports. The change in spanning arrangement means that the supports
511 closest to the removal location take up the loads that were previously taken from
512 the lost support and a higher proportion of the load on the alternate bay, as shown
513 by the decrease in forces at the further locations in Figures 6 and 22. This increase,
514 potentially more than 50%, might therefore exceed the shear capacity of the slab
515 and lead to a catastrophic failure. Furthermore, simple techniques for analysing
516 moment distributions for flat slabs, such as the equivalent frame method, can not
517 be applied after a column loss.

518 Increased loading, leading to further damage, does change the distribution of
519 forces slightly due to large rotations, changes in effective span lengths and a local
520 reduction in stiffness after cracking. However, with continuous slabs and restraint
521 provided by columns, these effects will be less significant and so static conditions
522 with small loading may provide suitable information to predict the final demand
523 on the supports.

524 *5.2. Whole slab behaviour*

525 The damage profiles, and results from the two penultimate cases, suggest that
526 the inclusion of continuous reinforcement through a column location does change
527 the distribution of stresses around the removed location. However, there is not
528 a significant difference in ultimate capacity. This is due to the change in load
529 paths away from the removed column. The static tests show that even after crack-
530 ing has occurred in the concrete and the reinforcement has yielded, the structure
531 can maintain its integrity and show a ductile behaviour. This is partly due to the
532 strain hardening in the steel reinforcement along with geometric nonlinearity as
533 the slab forms a tensile membrane at higher deflections, typically when the peak

534 displacement exceeds half the slab depth. However, the tests emphasised that
535 brittle mechanisms need to be avoided. A particular weakness of flat slab sys-
536 tems appears to be shear failure at corner supports. The additional demand placed
537 on these locations when a neighbouring column is lost, combined with their small
538 shear perimeter, makes them susceptible to progressive failures. Increasing punch-
539 ing shear capacity and ensuring surrounding supports have sufficient ductility can
540 therefore prevent progressive collapse.

541 Furthermore, although it seems that continuous bottom reinforcement through
542 a column may not be significant for flexural capacity, previous research has demon-
543 strated its efficiency in increasing the post-punching shear capacity of the sur-
544 rounding supports [23]. Therefore, its inclusion will aid in preventing progressive
545 shear failures.

546 For a flat slab structure, the global response of the surrounding elements plays
547 a key role as loads are redistributed due to the damage in the slab elements. In
548 these test the adjacent bays acted to counterbalance the damaged area leading
549 to lower deflections. Additionally, the continuous slab condition in tests C-S,
550 C-D and M-D allowed the formation of plastic hinges, which dissipated energy
551 from the system. However, in some cases plastic deformations continued after
552 the test, as shown by peak to final displacement ratios less than 1, which could
553 potentially lead to a later collapse. As the aim of these tests was to investigate the
554 general behaviour of slab elements to validate more detailed numerical modelling,
555 the inclusion of simple supports and non-fixed edges is not considered to be an
556 issue. However, further testing on realistic structural arrangements, including the
557 restraint provided by columns, is required.

558 *5.3. Dynamic effects*

559 The dynamic effects involved in suddenly removing a support can play a sig-
560 nificant role in the structural performance of flat slab structures. At low levels
561 of loading, within the elastic limits, there is typically a strong peak in deflections
562 followed by high frequency oscillations until the slab returns to rest after 3 or
563 4 seconds. At larger levels of loading, the additional mass increases the inertial
564 effect leading to a higher peak and more damage than from a static equivalent.
565 However, the damage also dissipates energy from the system via crack forma-
566 tions and plastic deformations of the steel, resulting in a lower frequency response
567 which is damped out within a second or two. Furthermore, after a sudden removal,
568 forces are not redistributed to surrounding supports instantaneously, with the peak
569 demand occurring as the structure comes to a temporary rest position between os-
570 cillations. Therefore, flexural damage may occur before a potential shear failure
571 and create a different response to the static loading case.

572 Typically in design cases, a factor of 2.0 is applied to the loading in the bays
573 around the removed column during a static analysis to account for dynamic ef-
574 fects. This is based on the behaviour of a linear elastic system with no damping
575 and instantaneous removal and theoretically represents the worst case scenario.
576 However, as all real structures experience some level of damping, it is clear this
577 amplification factor does not reflect a realistic condition. Furthermore, after crack-
578 ing occurs in the slab there is a reduction in its stiffness creating a nonlinear re-
579 sponse. Therefore, at common levels of loading, there is not a direct relationship
580 between the load applied and the level of displacement or damage. This is sig-
581 nificant because all observed force factors were considerably less than 2, though
582 further investigations are required to quantify this for typical structures.

583 The rate of the straining of the steel reinforcement from all the tests indicates
584 that the maximum strain rate is less than 0.35s^{-1} . However, this only occurs at very
585 localised points, which were undergoing significant plastic deformations already,
586 generally the strain rates in the steel were much less than this. High strain rates
587 change the material properties, most significantly increasing the tensile capacity
588 of concrete. To account for this the current Model Code [38], recommends a two
589 phase model, with a higher sensitivity after 10s^{-1} , for calculating the Dynamic
590 Increase Factor (DIF) for concrete due to fast loading.

591 Using the measured strain rates, the peak DIF the Model Code is 1.26, how-
592 ever, the results demonstrate that at the time of high strains, and therefore stresses,
593 the strain rate is fairly low. This is similar to the results from Yu et al. [39] in their
594 experimental investigation of RC beams under a sudden column loss. They mea-
595 sured strain rates of between 10^{-2} to $10^{-1}/\text{s}$, and concluded that this only gives a
596 small increase in material strength and can be conservatively ignored. This sug-
597 gests that the DIF for concrete may not be critical in providing additional flexural
598 capacity.

599 **6. Conclusions**

600 From the above results and discussion, the follow key conclusions can be
601 drawn.

- 602 • The sudden column loss idealisation can be reproduced on an experimen-
603 tal substructure of a flat slab floor. The use of a high speed camera with
604 image tracking can monitor deflections for the areas of interest during a dy-
605 namic removal condition and capture the formation of cracking. Although
606 true response of a slab structure is dependent on the surrounding elements,

607 a suitable substructure can provide useful information into the key perfor-
608 mance parameters.

- 609 • The ability of flat slab structures to efficiently span in two directions pro-
610 vides effective alternative load paths after a single column loss. Flexural
611 cracking was observed, both in the sagging areas and hogging over adjacent
612 columns, however, this did not lead to ultimate failure. All observed failures
613 were due to punching shear, usually at corner locations. Progressive shear
614 failures also occurred.
- 615 • A reduction in the stiffness of the flat slabs was observed at peak deflections
616 between 0.1 and 0.15 times the slab depth. However, beyond their elastic
617 limit, slab elements can still have significant additional capacity due to ma-
618 terial and geometric nonlinearities. As they enter the nonlinear range, there
619 is also a change in the response of the system. Force distributions change
620 and the damage alters the dynamic response of the system. Therefore, to
621 assess the true potential for a progressive failure these effects must be con-
622 sidered.
- 623 • The column loss event is inherently dynamic and the level of loading changes
624 the response of the system. This is due to two effects; the increase in mass
625 changes the natural frequency of the system and higher loading results in
626 damage to the structure. When damage occurs the dissipation of energy
627 affects the peak displacement and level of damping, as well as reducing
628 the stiffness, and therefore natural frequency. Additionally, a maximum
629 increase in displacements of 50% more than the static case was observed
630 during elastic tests due to inertial effects. This may therefore cause dam-

631 age to a structure near its limit, however this effect is less pronounced as
632 the structure experiences permanent damage. Common design recommen-
633 dations of a load increase of 2.0 appear to be conservative, especially con-
634 sidering the nonlinear relationship between force and displacements after
635 cracking. Furthermore, although high strain rates are known to increase the
636 material strength, the extent of straining and the time profile mean these
637 effects are less significant in assessing the progressive collapse potential.

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