Influence of dynamic effects on a concrete flat slab after a sudden column loss

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ABSTRACT: Prevention of a progressive collapse event of a structure is a key design consideration. Commonly a sudden column loss scenario is analyzed to ensure the structure has suitable robustness; however, there is still a lack of experimental information regarding the influence of the dynamic effects involved.

In this investigation 3 series of tests were conducted to investigate the dynamic response of a reinforced concrete flat slab structure after a column loss and to compare the damage caused by the inertial effects with a static push down test. In each test a substructure of a reinforced concrete flat slab at 1/3 scale was constructed and a UDL applied. For the static test a column was removed and the load increased. This was compared to a slab in which the full load was applied and the support suddenly removed and the system allowed to deform until it reached equilibrium or complete failure. Displacements were recorded during the tests and a high speed camera used to capture the dynamic motion.

The experiments demonstrated that flat slab structures are able to redistribute their loading effectively after a column loss. Whilst 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 disproportionate role the immediate columns play in transferring loads. The response of a structure to the loss of a column is a dynamic event with the level of loading and extent of damage influencing the response. Increases in deflections of 1.3 – 1.5 were observed due to inertial effects, however this is far less than the Dynamic Amplification Factor of 2.0 commonly applied to the loading indicating this may be over conservative.

KEY WORDS: Progressive collapse; Column loss; Flat slabs; Shear failure.

1 INTRODUCTION

The response of a structure after an initial damaging event is a critical consideration. This is reflected in the Eurocodes basic requirements that:

‘A structure shall be designed and executed in such a way that it will not be damaged by events such as explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause.’ [1]

One option for fulfilling this is to ensure the design can survive the accidental removal of an individual member; this has led to the use of the sudden column loss scenario in which a key vertical element is removed and the structure analyzed to predict if further failure is likely.

This situation has been investigated by a number of authors to determine the failure mechanisms and ultimate capacity of a damaged structure. These have included experimental tests on frame structures, both steel [2] and Reinforced Concrete (RC) [3-6]. This work has demonstrated the significance of nonlinear effects, both material and geometric, in providing additional capacity and preventing progressive failures. However sufficient ductility is required to allow yielding and the formation of catenaries else brittle failures may occur. These effects have also been considered numerically [7-11].

Further considerations have been given to the presence of slab elements which have been shown to increase the capacity of a structure after a column loss [12, 13]. RC slabs have complicated behavior at high loading due to their two dimensional nature allowing formation of tensile and compressive membranes [14-17]. Furthermore, their susceptibility to brittle mechanisms such as punching shear [18-22] may potentially lead to progressive collapse.

Progressive collapse is also a dynamic issue and suitable account needs to be taken of the inertial effects involved after a sudden damaging event. This can be done by conducting a full dynamic analysis of the structure; however this is time consuming, expensive and requires experience in order to achieve accurate results. Alternatively, an equivalent static case can be considered with a Dynamic Amplification Factor (DAF) applied to the loading. Current design recommendations suggest a DAF of 2.0 [23]. The suitability of this value has been studied for some structural types but further investigation is need for flat slab construction [10, 24-28].

This study provides much needed experimental evidence for the behavior of a flat slab structure after a sudden column loss. Three scaled slab models were investigated to simulate different column loss location scenarios and different levels of loading considered to observe the changes in the dynamic response. Dynamic results were compared to static tests to assess the additional damage sustained due to inertial effects. Details regarding the redistribution of forces and the damage patterns after an extreme event provide an indication into the potential for collapse of the structure.

2 EXPERIMENTAL PROCEDURE

2.1 Method

To investigate the behavior of an in-situ reinforced concrete flat slab structure, 3 series of 1/3 scale slabs were constructed.
These allowed simulation of the removal of a corner, penultimate edge or an internal edge column.

Two types of tests were conducted, an increase in static loading and a sudden dynamic column removal. Under the static case, the slab was placed on the supports and the column position under investigation was removed. A uniform force was then imposed across the entire sample by means of sand bags and the slab allowed to deform under the load during which force readings, deflections and cracking patterns were recorded.

Before each test, whilst the slab still had all supports, the vertical reaction forces were recorded at each location. This allowed the change in force distribution to be determined.

Under the dynamic removal, a similarly designed slab cast at the same time as the static, was loaded with sand bags whilst fully supported. Once the required UDL was achieved the chosen support was removed and the response recorded, primarily with a high speed camera.

2.2 Test setup

For the first series of tests a 2x1 bay subsection of a flat slab structure was constructed. The specimens were 4100mm x 2100mm in plan with a depth of 80mm. Each sample included A142 mesh providing 6mm bars at 200mm spacing for both top and bottom reinforcement. Additional 6mm bars were added over internal supports. This setup was used to replicate a corner or penultimate column loss as shown in Figure 1.

The middle column removal case used a 4x1 bay system, constructed in the same manner, with a total length of 8100mm (Figure 2).

Columns were simulated by 135mm square steel plates on bearings to allow rotations, each support also included a load cell to measure reaction forces. An example of the test set up is given in Figure 3 showing a fully loaded sample prior to the sudden removal of the front middle support.

The details of each test type, including concrete data taken from samples, are given in Table 1.

Linear Variable Displacement Transducers (LVDTs) sampled at 250Hz were used to measure vertical displacements at areas away from the column loss location. For the critical areas a Phantom V12.1 high speed camera capturing images at 2500fps allowed dynamic deformations to be recorded. Based on the camera resolution, target distance and imaging software, displacement readings were determined to be within ±0.1mm. Additional video imaging was used to monitor the static deformations at the key areas.
3 RESULTS

3.1 Corner column removal

Figure 4 shows the change in distribution of reaction forces for tests C-S and C-D. The change in distribution of force is calculated from the ratio of load each support takes between the damaged condition and its original fully supported value. The results at each load step were averaged to give the mean changes.

The distribution of force to each support did not change significantly as the total load was increased, indicating that the sand bags were placed uniformly across the surface. This also suggests there are only two structural systems experienced, the fully supported and the damaged case, and that even high levels of damage do not result in a change in load paths. There is also a good agreement between the specimens tested, indicating that sudden changes in support conditions do not lead to a 3rd structural system.

It is clear there is an increase in demand placed on the two adjacent columns, with an increase of between 41-57% whilst all other supports have decreased.

At high loading significant cracks formed due to the large increase in hogging moments in both tests, primarily on the top surface (Figure 5). Solid boxes indicate permanent support locations while the outline box is the location of the removed support. Sagging cracks also formed on the underside as the slab now spanned diagonally.

Displacements from the static increase in load for the corner loss condition are shown in Figure 6.

In this condition all displacements show a linear response at low load, however after 4.55kN/m² cracks start to form resulting in a decrease in stiffness to around 57% of the initial. At 6.04kN/m² there is a discontinuity due to significant cracking over the adjacent support along with yielding of the reinforcement. This leads to an increase of displacements across the entire sample, with the maximum exceeding half the slab depth. After this, there is a brief stiffening period before a final softening with a relative stiffness of 6% of the elastic range. The slab continues to carry additional load until the test was aborted at 8.2kN/m².

Displacements against time for different loadings at removal location after a corner column loss – Test C-D

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Table 2. Results from dynamic removal – Test C-D

<table>
<thead>
<tr>
<th>Loading (kN/m²)</th>
<th>3.02</th>
<th>6.81</th>
<th>7.65</th>
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<tr>
<td>Normalized Peak</td>
<td>0.07</td>
<td>0.59</td>
<td>1.16</td>
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<tr>
<td>Amplitude / Peak (%)</td>
<td>60.7</td>
<td>7.36</td>
<td>11.91</td>
</tr>
<tr>
<td>Peak / Final Displacement</td>
<td>1.42</td>
<td>1.02</td>
<td>1.07</td>
</tr>
<tr>
<td>Damped Natural Frequency (Hz)</td>
<td>11.0</td>
<td>5.41</td>
<td>3.54/4.21</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>0.01</td>
<td>0.24</td>
<td>0.123</td>
</tr>
</tbody>
</table>

In comparison, Figure 7 shows the response of the same slab set up, but with a sudden removal of the support. Three different levels of loading are presented and the deflections at the column location are given, normalized against the slab depth. The frequency spectrum from a Fourier transform is also given for each of the load cases (Figure 8).

Dynamic response values, i.e. the ratio of peak displacement to final, significance of the amplitude and the dominant frequency, are given in Table 2.

As the total load increased there is a noticeable change in the response of the system. With a UDL of 2.47kN/m² there is no damage and therefore little dissipation of the energy leading to a very low damping ratio, whilst the low total mass results in a high frequency. Whilst at 6.81kN/m², when cracks are just forming, the damping ratio is 10 times higher. In Figure 8, as loading increases there is a lower dominant frequency due to the larger mass in the system combined with the damage the structure as sustained. For the first two cases there is a strong peak frequency, however, at 7.65kN/m² the large deflections and damage creates a two frequency response.

There is also a change in relative size of the oscillations as a result of damage. Within the elastic range the amplitude between the first peak and first dip is 60.7% of the maximum displacement, indicating the structure returns relatively close to its starting value. Additionally, the peak displacement is 1.42 times the final. Once permanent damage occurs both these ratios drop considerably as seen in Table 2.

Finally the sample was loaded to failure, which occurred due to punching shear at the two adjacent supports as shown in Figure 9.

3.2 Penultimate column removal

For the loss of a penultimate column, Figure 10 shows the load-displacement plots at 4 locations across the sample for the static case. As seen in the corner condition above there is a linear response at low loads at all points, however once the slab experiences cracking and yielding at 3.93kN/m² there is a significant reduction in its stiffness, 3.8% of the initial value, and extensive further damage is observed. In this case, as the slab edges were not restrained, there was no formation of plastic hinges over supports, nor could significant membrane effects develop, there was a linear relationship ($R^2=0.995$) up to 6.45kN/m², when the test was ended.

The slab finally failed due to sudden shear failures at the two adjacent corner supports, as shown in Figure 11.

Under the dynamic condition the deflections against time at 3 locations for a UDL of 5.57kN/m² are shown in Figure 12. At the removal location there is a strong peak in deflection followed by a low frequency oscillation until it quickly finds an equilibrium state. Also under this condition the back corner support experienced uplift, emphasizing further the change in distribution of loading to each support as a result of the column loss. This uplift however did allow the slab to rotate inwards and experience higher deflections than would have been possible had the supports been restrained.

After the support has been lost, the removal location and the back support have damped natural frequencies of 3.36 and 3.5Hz respectively. However, as the back support position becomes free it also vibrates at twice the primary frequency, as visible in Figure 13.

Due to the damage experienced, mainly due to sagging cracking on the underside, there are large deflections, with a peak value of 1.49 times the slab depth.
Table 3. Results from dynamic removal – Test P-D

<table>
<thead>
<tr>
<th>Loading (kN/m²)</th>
<th>2.48</th>
<th>5.57</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalized Peak</td>
<td>0.14</td>
<td>1.49</td>
</tr>
<tr>
<td>Amplitude / Peak (%)</td>
<td>44.3</td>
<td>6.82</td>
</tr>
<tr>
<td>Peak / Final Displacement</td>
<td>1.48</td>
<td>1.09</td>
</tr>
<tr>
<td>Damped Natural Frequency (Hz)</td>
<td>6.1</td>
<td>3.41</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>0.277</td>
<td>0.198</td>
</tr>
</tbody>
</table>

The different loading cases, shown in Table 3, indicate that although 2.48kN/m² was within the elastic range for the static condition, there is evidence that the slab has now experienced damage. The values of damped natural frequency and amplitude to peak ratio are noticeably lower than were observed in the other tests (Tables 1 and 3). Additionally the damping ratio and normalized peak are higher than would have otherwise been expected if damage had not occurred due to the inertial effects taking the material past its elastic limit. However, the dynamic factors are still much less significant with higher loading (5.57kN/m²).

### 3.3 Middle column removal

For the final test, M-D, support reactions were recorded and the change from the fully supported condition to the damaged are shown in Figure 14. The results for both left and right sides of the line of symmetry are given and show similar values indicating that the slab was loaded and deformed evenly. As seen before there is a high demand placed on the immediately adjacent supports whilst areas further away have a reduction.

Dynamic removal tests were conducted at different loadings and the deflections at the column location captured from the high speed camera are plotted in Figure 15.
While this condition shows the same behavior as the previous, there is a significant reduction in displacements. This is due to less rotation at the continuous supports, and the counterbalancing effect of the adjacent bays which also prevented uplift at the supports providing a more realistic condition. At the lower levels of loading the bays adjacent to the damaged area experience a slight uplift, as shown by the negative displacements in Figure 16. This is due to the slab rotating inwards towards the removed support. However, at higher loadings, there is a brief uplifting effect, label (A) in Figure 16, but the damage sustained across the slab results in a final downward trend.

In the later tests there was considerable cracking of both the top and bottom surfaces of the concrete. This led to large plastic deformations and the drift observed in Figures 15 and 16. However, collapse due to total flexural failure did not seem likely and shear cracks did not form within the levels of loads tested.

Table 4 gives the values of dynamic effect for three loading levels. As observed for the earlier tests, in the elastic range (3.10kN/m²) there is a high frequency response due to the low levels of mass, combined with a small damping ratio due to the little dissipation of energy via plastic deformation or concrete fracture. This case also shows the influence of inertial effects on a lightly loaded slab, with a high peak to final displacement ratio, 1.54.

As the loading increases there is a decrease in frequency response as would be expected, from 13.4Hz at 3.10kN/m² to 8.55 and 6.00Hz for 6.86kN/m² and 8.51kN/m² respectively, combined with similar increases in damping ratios due to the damage sustained.

Finally, the response of the adjacent bay (Figure 16) shows a similar frequency of oscillation for the elastic test, 13.3Hz. However at the highest load there is still a damped frequency of 10.4Hz despite a damping ratio of 0.17 compared to 0.02 in the earlier test.

Figure 17 shows a photograph of the underside of the slab after the test was completed with the cracks annotated. The primary cracking pattern is shown in black; here the two-way spanning nature of a slab structure after a column loss is clear by the diagonal cracks. The red lines are secondary flexural cracks that follow the reinforcement lines; these were more extensive than would be expected due to the slab not being continuous in both directions. The top cracking due to the increased hogging moments over the adjacent supports was almost identical to corner removal case shown previously in Figure 5.

**Table 4. Results from dynamic removal – Test M-D**

<table>
<thead>
<tr>
<th>Loading (kN/m²)</th>
<th>3.10</th>
<th>6.86</th>
<th>8.51</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalized Peak</td>
<td>0.05</td>
<td>0.09</td>
<td>0.24</td>
</tr>
<tr>
<td>Amplitude / Peak (%)</td>
<td>67.31</td>
<td>15.98</td>
<td>5.82</td>
</tr>
<tr>
<td>Peak / Final Displacement</td>
<td>1.54</td>
<td>&lt;0.92</td>
<td>&lt;0.90</td>
</tr>
<tr>
<td>Damped Natural Frequency (Hz)</td>
<td>13.4</td>
<td>8.55</td>
<td>6.00</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>0.017</td>
<td>0.219</td>
<td>0.204</td>
</tr>
</tbody>
</table>

Figure 15. Displacements against time for different loadings at removal location after a middle column loss – Test M-D

Figure 16. Displacements against time for the center of the adjacent bay after a middle column loss – Test M-D

Figure 17. Annotated underside cracking pattern for continuous slab – Test M-D
4 DISCUSSION

These tests sought to replicate the effect of a column loss on a flat slab system. Analysis of the high speed footage shows that the support was completely removed within 50ms. Since, for the tests at loads high enough to cause damage, the slab took at least 200ms to reach a maximum the removal method can be considered instantaneous or sudden.

The reaction force distribution and the cracking patterns shown in Figures 4, 5, 14 and 17 give a good indication into the change in load paths that a damaged slab experiences. The bending profile becomes truly two-dimensional, with new spans primarily acting diagonally between supports. Therefore, additional or continuous bottom reinforcement may not be the most effective means of increasing capacity. Furthermore, the distribution of forces remained similar between the two test types and at all loading levels, indicating that a static approach can be used in place of a dynamic analysis.

There is also a disproportionate extra demand placed the supports closest to the removal location. Not only do they transfer the loads that were previously taken from the lost support, the change in spanning arrangement means they take a higher proportion of the load on the alternate bay, as shown by the decrease in forces at the further locations in Figures 4 and 14. This increase of potentially more than 50% may therefore exceed the shear capacity of the slab.

The results from the static tests show that even after cracking has occurred in the concrete, along with yielding of the reinforcement, the structure can maintain integrity and show a very ductile behavior. However, the tests did emphasize that brittle mechanisms need to be avoided. Increasing punching shear capacity and ensuring surrounding supports have sufficient ductility can therefore prevent progressive collapse.

It is also clear that the dynamic effects involved in suddenly removing a support can play a significant role. At low levels of loading, within the elastic limits, there typically is a strong peak in deflections followed by high frequency oscillations until it returns to equilibrium after 3 or 4 seconds. At larger levels of loading the behavior changes, the additional mass increases the inertial effect leading to a higher peak and more damage than from a static equivalent. However, the damage also dissipates energy from the system via crack formations and plastic deformations of the steel. This results in a lower frequency response which is damped out within a second or two. In some cases though, the damage the peak caused resulted in additional drift occurring which lasted much longer.

The results also highlight the influence of the surrounding bays in preventing progressive failures. Firstly, the adjacent bays acted to counterbalance the damaged area leading to lower deflections. Additionally, the continuous slab condition in tests C-S, C-D and M-D allowed formation of plastic hinges which dissipated energy from the system, however, the plastic deformations did continue for a while after the test in some cases, as shown by peak to final displacement ratios which were less than 1. This effect could potentially lead to a later collapse.

Considering the two test types show that at low levels the final dynamic deflections are comparable to the equivalent static type. This is due to the system remaining in the elastic range even after the inertial effects increasing the peak deflection by between 30-50%. However at higher loads larger permanent deflections and additional damage can be ascribed due to the dynamic influence. For the corner location there was a peak displacement of 1.35 times the value measured for a static test. This then lead to a final displacement of 1.26 times the static.

Typically a factor of 2.0 is applied to the loading around a the damaged area during a static analysis to account for dynamic effects [23], however, after cracking occurs in the structure there is a reduction in its stiffness creating a nonlinear response. Therefore, at common levels of loading, there is not a direct relationship between the load applied and the level of displacement or damage. This is significant as an increase of deflections far less than 2.0 was observed, which corresponds to an even lower force factor.

These tests alone cannot provide detailed information into the exact amplification factor that the dynamic case creates, however the results indicate that whilst inertial forces do increase the damage the structure will sustain and could potentially lead to progressive failures, this effect is far less than the commonly used factor of 2.0. Further numerical investigations will be conducted to consider this effect and determine suitable values for the DAF for flat slab structures.

5 CONCLUSIONS

Considering the results and discussion the follow key conclusions can be drawn.

1. A scaled substructure can be used to investigate the behavior of a flat slab after a damaging event, such as a column loss. However, the surrounding bays play a significant role in both the response and the ultimate capacity and therefore should be suitable included. Use of high speed monitoring methods also allows the response from a sudden removal to be studied accurately.

2. Flat slabs have a complex behavior after a loss of a support which includes its two directional spanning nature as well as material and geometric nonlinearities. These effects can provide alternative load paths and capacity which prevent further failures from occurring.

3. As a result of the alternative load paths, flat slabs can show a ductile behavior when a support is removed as long as the flexural reinforcement provides adequate integrity. However, brittle failure such as punching shear is a concern and designs should focus on preventing these mechanisms to protect against progressive failure.

4. The column loss event is inherently dynamic and these effects need to be considered. The response changes from oscillation with a sharp peak and high frequency in the elastic range to a more gradual response that is quickly damped out once damage occurs. This is partly due to the increase in mass of changing the natural frequency along with this dissipation of energy via concrete cracking and plastic yielding.

5. The increase in deflections as a result of the dynamic removal plays an important role in determining the potential for progressive failure. A peak increase of
50% more than the static case was observed during elastic responses, this may therefore cause damage to a structure near its limit. This effect though is less pronounced as the structure enters the nonlinear range and experiences damage. However, design recommendations of a load increase of 2.0 are very conservative, especially considering the nonlinear relationship between force and displacements after cracking.

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