STRUCTURE ROBUSTNESS AGAINST PROGRESSIVE COLLAPSE DUE TO UNFORESEEABLE EVENTS

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A robust structure will survive and remain serviceable during its design life where unforeseeable events or attacks are likely to occur. Examples of such unforeseeable events are: change of use; sudden and large settlements of supports; extreme impact; war; fire; extreme wind; catastrophic events, etc. Two case studies are presented in this paper, a multi-story flat plate building subjected to column loss and a cable-stayed bridge subjected to cable(s) loss, in order demonstrate such a role of robustness. The finite element method considering both material and geometric nonlinearity, with the aid of the software ABAQUS, is employed in this study to perform both static and dynamic analysis of the relevant structures. The two examples reveal how robustness can prevent progressive collapse of structures due to unforeseeable events. The flat plate building with continuous bottom reinforcement could survive column loss. The system robustness enabled the cable-stayed bridge to survive cable(s) loss.

Keywords: Finite Element, Flat Plate, Cable-Stayed Bridge, Column loss, Cable loss.

1 INTRODUCTION

Robustness is an undefinable resistance and hidden quality beyond a defined safety or durability according to codes and standards. A robust structure will survive and remain serviceable during its design life where unforeseeable events or attacks are likely to occur. Examples of these events are: change of use; sudden and large settlements of supports; extreme impact; short term overloading during erection; catastrophic events; sabotage; war; fire; extreme wind; ..., etc. (Schlaich and Pötzl 1992). The probability of occurrence of such unforeseeable events is extremely small and their type and intensity can only, if possible, be recorded in a very broad sense; besides, they are not addressed in standard codes of practice. Therefore, such events are usually neglected in the dimensioning process and hence forgotten in the design. The space frame of the Hartford Civic Center in the United States collapsed in 1978 due to heavy snow (Fu 2016). A recent example is the progressive collapse of the suspension bridge Kutai Kartanegara in East Borneo, Indonesia (Fu 2016).

In this study two different examples of structures are examined in order to show how robustness can play a role in preventing progressive- or disproportionate-collapse due to unforeseeable events. These examples are a multi-story flat plate building subjected to column loss; and a cable-stayed bridge subjected to cable(s) loss. The investigation has been carried out through conducting three-dimensional nonlinear static and dynamic finite element analyses using the software (ABAQUS
(El-Demerdash 2019).

2 CASE STUDY 1 – COLUMN LOSS IN FLAT PLATE MULTISTORY BUILDING

2.1 Building Description

Compared with modern construction, older flat plate structures in the United States without continuous slab bottom reinforcement at columns until 1971 (ACI 318-71) did not inherit structural integrity after a punching shear failure is initiated at a slab-column connection, thus this system is deemed more vulnerable to progressive collapse. Generally, as per the ACI 318-14, part of the longitudinal bottom reinforcement in the column strip is required to continue through the column in order to prevent progressive collapse. Tensile membrane action in this system may become the main load redistribution mechanism after a column loss.

The finite element is employed to carry out both static and dynamic analysis in order to evaluate the response of both an older (ACI 318-71) and a modern (ACI 318-14) four bays-four stories reinforced concrete flat plate office building, Figure 1, subjected to an instant removal of column. Each building was analyzed for gravity loads with the simulation of the loss of: an interior column; an exterior column; and a corner column, each at a time. In this building, all the columns are square, 380mm width, and all floor slabs have the same thickness of 191mm. The service loads acting on the floor slabs include a live load of 2.40kN/m² and a dead load of 5.44kN/m². Grade 60 reinforcement with a yield stress $f_y = 414$ MPa and a concrete of a cylinder strength $f'_c = 27.6$ MPa are assumed for all structural elements. The individually lost columns are C3, C5, and A5 at the first story, Figure 1.

![Figure 1. Flat plate building: (a) 3D view; (b) plan.](image)

2.2 Simulation Procedure of Column Loss

The loss of a column due to an explosion, for instance, is a dynamic process; therefore, this effect can be modeled by replacing the potential failed column in Figure 2a with its reaction R, Figure 2b, and an equivalent opposite point load at the top of the column position, Figure 2c. At first, the gravity loads accompanied with the equivalent column reaction, W and R, are applied gradually to their full amount during 0.5 second and kept unchanged for another 0.5 second to avoid dynamic effect at this stage, Figure 2d. Then, the column reaction is released within a period of 0.005 second, Figure 2d, by adding the downward load R at the top of the column position. This duration
of column removal is based on the requirement of the Department of Defense regulations (DoD 2009), when nonlinear dynamic analysis procedure is employed, this shall be less than 0.10 times the natural period associated with the vertical vibration of the bays above the removed column, which is 0.335 second for this case. In addition, from the field test of a 10-story concrete frame structure (Sasani et al. 2007), the time spent on column removal by explosion was about 0.005 second. Afterwards, the time history of the gravity loads is extended for additional 2.0 seconds.

\[ \text{Figure 2. Simulation of column loss by fictitious opposite forces and the Load- time curves for dynamic progressive collapse analysis procedure.} \]

### 2.3 Results

- The failure mode of an older flat plate building without continuous slab bottom reinforcement (upon the loss of interior, exterior or corner column, each at a time) consisted of punching shear failure accompanied with rupture of the slab reinforcement. As the floor slab cannot develop sufficient tensile membrane action to carry gravity loads, the entire building was prone to experience progressive collapse, Figure 3.

\[ \text{Figure 3. The older flat plate building - the vertical deflection in 1st floor above the lost column.} \]

- In the modern building, the Vierendeel action was the primary mechanism for load redistribution because of the presence of the continuous bottom reinforcement in column strip, which made the building to work as one unit, Figure 4. Thus, bottom reinforcement continuity plays an important role in limiting the deformation of the system in comparison with the deformation of
the older building, in the case of column loss, Figures 3 and 4. When the interior column C3 failed, its load was redistributed to the neighboring columns C4, C2, D3, and B3, and no progressive collapse happened. Similar response took place when removing the exterior column C5 or the corner column A5. Losing an exterior or a corner column was particularly critical where the potential of progressive collapse was relatively large.

3 CASE STUDY 2 – CABLE LOSS IN CABLE-STAYED BRIDGE

3.1 Bridge Description

Cable-stayed bridges are subjected to corrosion of connections, vehicle impact, wind, and fatigue which may result in a reduction in the cross-section and in their resistance or cable loss. Cable loss can lead to overloading and rupture of adjacent cables. The collapse in such a way is called the zipper-type progressive collapse because it happens to the entire bridge. Therefore, cable-stayed bridges should be designed for possible loss of any single cable and the loss should not lead to the immediate failure of the entire structure as recommended by some guidelines (PTI 2007).

![Diagram of Quincy Bayview bridge](image.png)

Figure 5. Quincy Bayview bridge (Wilson and Gravelle 1991): (a) front elevation; (b) actual deck cross section; (c) idealized deck cross section for modeling; and (d) Pylon section.

A three-dimensional finite element analysis via the software ABAQUS (2013) has been performed in order to predict the nonlinear dynamic response under an instantaneous removal of one or more cables of the Quincy Bayview cable-stayed bridge, located in Illinois, USA. The effect of number of lost cables and their locations has been examined with the objective of defining the most critical cables on the potential collapse of the bridge. In cable-stayed bridges, there are three sources of geometric nonlinearities, namely; large displacement, P-Δ and cable sag effects; therefore, the effect of geometric nonlinearity has been accounted for by modeling the stay cable as a 3D beam element. The Quincy Bayview Bridge (Wilson and Gravelle 1991), consists of two H-shaped concrete towers, double-plane fan type cables, and a composite concrete-steel girder bridge deck. The main span is 274.32m and there are two equal side spans of 134.11m making a total bridge length of 542.54m, Figure 5a. The bridge is described in Figure 5 and all details are given in (Wilson and Gravelle 1991).

3.2 Analysis

For each case of cable loss, the bridge has been analyzed under dead loads in addition to three cases
of traffic loads: in the full span, in the middle span only and in the outer spans only. The traffic live loads are those of the (AASHTO 2012) specifications which allow to represent the truck as a single concentrated load of 325kN and a uniform load of 9.3kN/m/lane (3.1kN/m²). The different scenarios of cable loss considered in this study are given in Table 1. The cables were prestressed through connection to the deck under the action of dead load (self-weight). Hence, the main girder is assumed not to deflect at the cable anchorage points and the moment distribution of the girder corresponds to that of a continuous beam on rigid supports for the case of self-weight. For this case of loading, the cable prestress level is approximately one-third its ultimate tensile stress (Fu 2016); therefore, a value of initial stress 528MPa was assumed in model.

Table 1. Scenarios of cable loss.

<table>
<thead>
<tr>
<th>Scenario</th>
<th># of cables (Figure 5)</th>
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<tbody>
<tr>
<td>Scenarios of one cable loss</td>
<td>1; 8; 14; 15; 28</td>
</tr>
<tr>
<td>Scenarios of two cables loss</td>
<td>14, 15; 1, 2</td>
</tr>
<tr>
<td>Scenarios of three cables loss</td>
<td>12, 13, 14; 1, 2, 3</td>
</tr>
</tbody>
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Figure 6. Modeling the cable loss using fictitious opposite loads.

The sudden loss of a cable has been modeled in the same procedure followed in the previous case study, Figure 6. In the 3D nonlinear dynamic finite element analyses carried out for this example, the bridge deck and the pylon strut web were modeled by four-node continuum 3D shell element S4R. For the steel of the main girders, cross girders, and stringers, two-node linear 3D beam elements were used based on gross cross-section properties. The term linear in element designation refers to linear interpolation function. Two-node linear 3D beam elements were used for modeling the concrete pylon (tower) based on the gross cross-section properties. The cables were modeled as nonlinear quadratic (second order) beam elements (B32 in ABAQUS element library). For the static analysis, following the Department of Defense regulations (DoD 2009), a service gravity load W consisting of 1.20 times the dead load plus 0.50 times the live load was statically applied to the deck floor slabs.

3.3 Results

- The loss of either a single cable or two simultaneous cables led to significant bending moments in the main girder and significant stresses in the adjacent cables but did not cause collapse since the cable stresses did not change significantly and remained within their yield strength.
- The loss of any simultaneous three cables yielded high stresses in the adjacent cables such that these cables yielded; subsequently snapped and a zipper-type collapse progression has been triggered, Figure 7. If such a loss occurs in the main span, it would cause yielding of the cables adjacent to those lost and a large excessive vertical deformation within the bridge deck followed by a zipper-type collapse progression.
Figure 7. Samples of cable stresses for case of simultaneous loss of three cables.

- For the case of total and truck loads in the main span, the longest main stay cables were the most critical in terms of maximum vertical displacement in the deck, bending moment in the main girder, and stresses in the adjacent cables for either case of cable(s) loss. However, the longest backstay cables were the most critical in terms of the lateral deformation of the pylons.
- The possibility of failure progression in the model, decreased if the failed cables were closer to the pylon or abutment. This can be attributed to the minor increase of the forces in the adjacent cables such that they did not reach their yield strength since great portions of the forces of the lost cables were transferred to the pylon or abutment.

4 CONCLUSIONS

The two studied cases presented in this paper reveal how robustness can prevent progressive collapse of structures due to unforeseeable events. The flat plate building with continuous bottom reinforcement could survive column loss. The system robustness enabled the Quincy Bayview Bridge to survive cable(s) loss.

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