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Progressive collapse evaluation of Murrah Federal Building following sudden loss of column G20

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ABSTRACT

The Murrah Federal Building (MFB) was the main target of the Oklahoma City Bombing in 1995. Previous studies have concluded that the building would have collapsed even if exterior column G20 was statically removed. In this paper, the system-level response of the MFB due to the sudden loss of column G20 is analytically studied. It is demonstrated that the building would have resisted progressive collapse, even if the column was suddenly removed. Two important reasons have led to a different conclusion from those of the previous studies. First, the axial compressive force of the column above the lost column diminishes only a few milliseconds after column removal, thus, it does not continue to push the supporting girder down. Second, two collapse resisting mechanisms were not considered in the previous studies: (a) a beam's tendency to grow as it cracks and yields under flexure and its effects on the axial–flexural response of the 3rd floor transfer girder, resulting in the enhancement of its gravity load carrying capacity and (b) the redistribution of the gravity loads through new load paths in both longitudinal and transverse directions through Vierendeel frame action. Given that the structure collapsed, the initial damage to the MFB must have been more severe than a sudden loss of column G20.

1. Introduction

On April 19, 1995, an explosion of a truck loaded with 4000 lb (1812 kg) of equivalent TNT caused collapse of almost half of the MFB and death of 168 people [1–3]. The attack on this building remains the most destructive act of terrorism committed in the United States prior to the terrorist attacks on September 11, 2001. Fig. 1 shows the north view of the building before collapse and Fig. 2 shows the 3rd floor plan of the building, the location of columns, Transverse beams (T-beams), longitudinal Transfer Girder (TG), and the bomb crater. The 2nd floor columns supporting the TG are also shown by hollow rectangles.

Two primary sets of publications which evaluated the collapse of the MFB are: (a) the FEMA/ASCE report [4] and the paper by Sozen et al. [5] and (b) the ASCE report by Hinman and Hammond [1] and the paper by Osteraas [3]. There are other studies that analytically evaluated the collapse of the MFB. Tagel-Din and Rahman [6] used the Applied Element Method to simulate the collapse process of the MFB. They presented the real time response of the building, failure of column G20, and the failure of the TG till partial collapse of the structure. Byfield and Paramasivam [7] introduced a method of predicting column failures due to blast and used the method to evaluate the column failure pattern reported during the forensic investigation of the MFB. They replaced the 3rd floor TG with a conventional beam–column arrangement (i.e. all the columns on line G were continuous to the ground) and adjusted the longitudinal exterior frame on line G, accordingly. Using the failure pattern of the reconfigured building they concluded that the extent of the collapse would have been largely unchanged.

The purposes of the investigation by the FEMA/ASCE report [4] and Sozen et al. [5] were to evaluate the initial damage caused by the blast, determine the failure mechanism for the building, and review design methods for reducing likelihood of collapse of buildings in the future. The nominal flexural strengths of the girders' sections of the line G frame were determined. Then two-dimensional plastic analyses of the structure were carried out and the possibility of the collapse of the line G frame was determined. They concluded that even a static removal of column G20 on the 1st and 2nd floors would create sufficient reason for structural collapse of column line G between column lines 16 and 24. A brief explanation of the failure mechanism along with the calculation method is presented in the last section of this paper.

This paper reevaluates the above mentioned conclusion by accounting for the load redistribution process and additional

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Fig. 1. North view of MFB before collapse (courtesy: John D. Osteraas).

collapse resisting mechanisms. In this study, a system–level three-dimensional numerical simulation of the structure is utilized to evaluate the response of the MFB to sudden removal of the 1st and 2nd floor columns G20. The primary resisting mechanisms of the structural system and the alternate load-transfer paths are identified. Then, the possibility of the collapse of the MFB due to the sudden loss of column G20 is investigated. Finally, a review of FEMA 277 [4] and the paper by Sozen et al. [5] is presented.

2. Building descriptions

The MFB was a nine story building located in Oklahoma City, constructed in 1974 [5], and designed according to ACI 318-71 [8]. The structural system of the building was an ordinary Reinforced Concrete (RC) moment frame. The lateral load resisting system for wind forces was comprised of RC shear walls located within the stair and elevator core on the south side of the building.

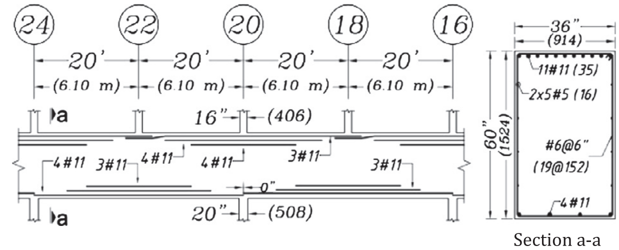

Fig. 3. Longitudinal profile of TG reinforcement, NTS (numbers in parenthesis are in m or mm).

The structural system of the 3rd floor included a TG in the longitudinal direction (E–W) at the north face of the building. The TG section was 36″ wide and 60″ deep (914 by 1524 mm). Fig. 3 shows a longitudinal profile of the middle portion and one cross-section of the TG reinforcement. As can be seen, the bottom bars of the TG were not continuous over the top of the main columns (i.e. the splice length was zero). Therefore, the #11 (36 mm) bottom bars were embedded by only 10″ (254 mm) into the joint ($7d_b$).

Columns below and above the 3rd floor on line G were spaced 40′ and 20′ (12.19 m and 6.10 m), respectively. The 3rd floor columns on lines 10, 14, 18, 22, and 26 were supported by the TG and were not continuous to the ground (see also Fig. 19 for column configuration). The typical story height was 13′ (3.96 m) and the height of the 9th story was 14′ (4.27 m). The cross sections of the 1st and 2nd floor columns on line G were 20″ by 36″ (508 by 914 mm) and those of the columns above the 3rd floor were 16″ by 24″ (406 by 610 mm). Fig. 4 shows the cross sections of the columns on line G. The columns on line F were 24″ by 24″ (610 by 610 mm) up to the 4th floor and 20″ by 20″ (508 by 508 mm) above it. The locations of the 3rd floor columns on line G, in the transverse direction (N–S), were not clearly mentioned in the available documents [4,5], and also could not be inferred from the drawings. In this study, considering the space required for the façade system and a photo of the building after collapse, it is assumed that the center of the 3rd floor columns on line G, the center of the 2nd floor columns on line G, and the center of TG were vertically aligned (see zoom–in detail in Fig. 2).

The floor system was 6″ (152 mm) thick one way slabs spanning in the longitudinal direction and supported by T-beams, which ran in the transverse direction. The slab top reinforcement in the longitudinal direction consisted of 12′ (3.66 m) long #4@16″ bars (13 mm @ 406 mm), and 10′ (3.05 m) long #4@16″ bars, both centered at the centerline of the T-beams. Fig. 5 shows the reinforcement of the T-beam and floor slab around column G20. Considering the 20′ (6.10 m) spacing of the T-beams and 12′ (3.66 m) length of

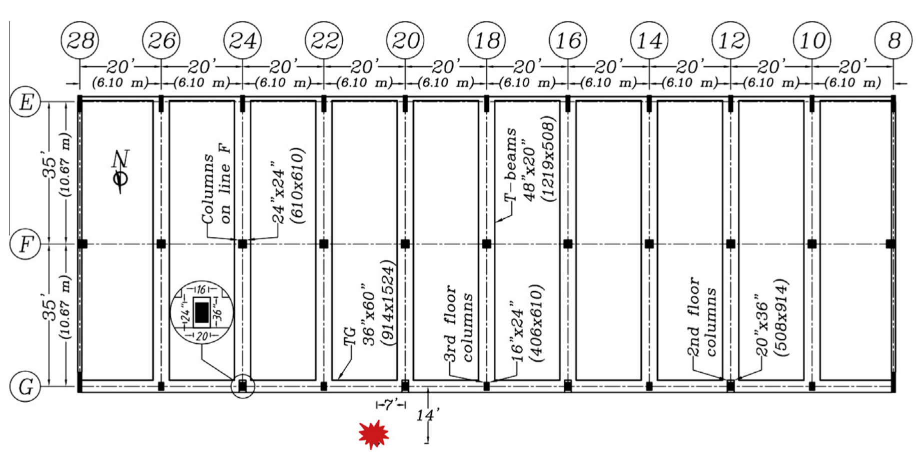

Fig. 2. Plan view of 3rd floor (numbers in parenthesis are in m or mm).