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ANALYSES OF STRUCTURAL RESPONSES AND DAMAGES OF WORLD TRADE CENTER TOWER 2 DUE TO IMPACT OF AIRCRAFT CRASHED

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Abstract
The September 11, 2001, attacks on WTC1 and 2 caused extensive structural and local damages at the points directly hit by the aircrafts. Despite massive local damages to floors, both towers remained standing for more than about one hour. Although the damages of beams and columns in the exterior perimeter of the towers were reported and published in the ASCE/FEMA report, the actual extent of the damage of the floor system and inner core columns were not actually known. A theoretical investigation was carried out to examine the structural response of the towers, which could help to provide some explanations on why the towers remained standing that long upon direct impact of the aircrafts. Some analyses were undertaken using a simplified model to estimate the overall damages caused while finite element analyses were used to examine local damages and stress conditions.

Keywords: WTC, Aircraft impacts, Global damage, Local damage, Finite element model, Gravity load analysis

1. Introduction
The progressive collapse of World Trade Center (WTC) towers in New York, USA on September 11, 2001 was an unprecedented event in the history of building construction. More amazingly was that the tower did not collapse immediately following the impacts but was able to remain standing for more than about one hour which made it possible for occupants of the south tower (WTC2) and north tower (WTC1) to evacuate. Many believed that the redundancy in design made the towers to survive the impacts and prevent from immediate collapse. A better understanding on redundancy and robustness needs to be pursued in generating a better understanding of the behavior of structures, especially when such design basis could offer a safer design alternative for occupants.

In the study to be described in the foregoing paragraphs, the overall response of WTC2 is estimated using a lumped mass model and comparison made with the design wind load. Damage to members is analyzed using a finite element technique.

2. Structural Design and Configuration of the WTC Towers
2.1 Structural design
WTC1 and 2 were 110-story office towers made of steel with seven basement floors. The buildings were 417 m and 415m high with the typical story height of 3.676 m. The buildings were 63.1 m x 63.1 m square in plan with beveled corners and a rectangular core measuring 26.5 m wide and 41.8 m long. The longer side of the core was in the east-west direction for WTC1 and the north-south direction for WTC2. The tubular structure comprised of 240 box columns installed along the perimeter at intervals of 1.016 m with steel spandrel beams carried the horizontal design load to be mostly wind load and the gravity load equivalent to about 40% of the building weight. The core comprised of 47 columns connected by H-shaped small beams. The columns were flat box-shaped in the lower floors and H-shaped in the upper floors. The core was designed to support only the gravity load equivalent to about 60% of the building weight. Floor slabs were steel decks with lightweight concrete supported by steel-framed truss beams which were pin-jointed to the perimeter columns and the core. Thus, the entire floor formed a diaphragm. From the 107th to 110th story, braces were installed forming outrigger truss systems, six in the north-south direction and four in the east-west direction (Fig.1).
Fig. 1 Floor plan and outrigger truss system of WTC2 tower

(b) Top of A×A section (Outrigger truss)

Table 1 Natural period of WTC2 Tower

<table>
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<th>Mode</th>
<th>Natural period (sec)</th>
</tr>
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<tr>
<td>1</td>
<td>13.0</td>
</tr>
<tr>
<td>2</td>
<td>4.62</td>
</tr>
<tr>
<td>3</td>
<td>2.68</td>
</tr>
</tbody>
</table>

Fig. 2 Analytical model for overall response estimation

Fig. 3 Velocity reduction curve

Fig. 4 Impact load time-history
The cross sections, layout and thickness of columns and beams were extracted from published materials. The thickness of cross section and floor plan of each floor were, however, unknown as no structural design drawings were made available. The cross sections and layout of structural members were assumed as described below based on detail reported in the ASCE/FEMA report (2002), Hart et al. (1977) and Tsuruta (1970).

2.2 Perimeter structure

2.2.1 Perimeter columns.
Box columns measuring 356 x 356 mm and consisting of four steel plates with thickness varying from 6-76mm were used. In the bottom six floors, one box column 813 x 813 mm was used instead of three columns in the upper floors. The distribution of plate thickness in each floor was assumed to provide a sufficient allowance for an average axial stress under gravity loads set at approximately 100 N/mm² against the yield strengths of 450 and 690 N/mm² for the types of steel used in the construction. The three-layer unit columns were connected to other units by bonding the end plates with 22-mm-diameter bolts with a maximum strength 830 N/mm². Four and six bolts were used in the upper and lower floors respectively.

2.2.2 Perimeter beams.
The steel spandrel beams were 1321 mm depth and 9 to 36 mm thickness. The yield strength was either 450 or 690 N/mm² depending on the material used. High-strength bolts were used for connection.

2.3 Inner core structure

2.3.1 Core columns.
Box columns 1350 x 610 mm were used below the 50th floor and 700 x 550 mm box columns were adopted below the 70th floor. Above 70th floor, extremely thick H-sections columns with maximum size is H-570 x 454 x 78 x 125 were installed. The cross section was designed so that the axial stress could nearly equal two-thirds of the yield strength of 250 N/mm² under gravity loads.

2.3.2 Core small beams.
H-sections H-459 x154 x 9 x 5 were mainly adopted.

2.3.3 Floor slabs.
Steel decks with a thickness of 1 mm and a groove depth of 38 mm were placed parallel to the main truss beams. Light-weight concrete of 102 mm thickness was used as floor slabs.

2.4 Weight and design wind load

2.4.1 Weight
The total building weight above ground was estimated at 3,630 MN (Tsuruta 1970). The dead and live loads were 2,890 and 740 MN, respectively. The mean weight per unit area was 8.29 kN/m².

2.4.2 Design wind load
The design wind load was 2.16 kN/m² on average, equivalent to a base shear coefficient of 1.6%.

3. Behavior of the Tower under Impact Load (Overall Response)

3.1 Model of the building
A three-dimensional frame model was built by representing the column and beam members on the perimeter of the tubular structure by general beam elements taking into effects of bending, shearing and axial deformation. Static design wind load was applied to the model and the resultant deformation being separated into overall bending and shearing deformation in order to assess the bending and shearing stiffness. A single-line model with 110 beam-lumped masses to represent 110 floors of the tower was developed (Fig.2). Each floor was assumed to weigh 33 MN. The natural periods of the model
are listed in Table 1. A Rayleigh damping of 3% was assumed for the fundamental and secondary modes of vibration, based on the measurements taken at high wind velocities (Mahmoodi 1987).

3.2 Impact load

The impact load was simulated through an analysis with the aircraft crash into a rigid wall. This will be described in detail in section 4.1. The time histories of impact loads that should be applied to respective mass points from 77th through 85th floors were generated from the reaction forces of the rigid wall corresponding to the floors hit by the aircraft. As the building and the aircraft were entirely damaged, the peak load was smaller than the rigid wall and the impact load lasted longer. Therefore, the impact load curve was modified as described below in an attempt to get a better representation of the actual forces.

According to the velocity reduction curve (Fig. 3), it took approximately 0.4 seconds for the nose of the aircraft to penetrate the building after the impact. The impact load was prolonged so that the duration time of 0.25 seconds in case of aircraft impact into the rigid wall would correspond to 0.4 seconds of actual impact. The reduction on load is to produce an equivalent impulse. The load obtained in the above process (Fig. 4) was applied to the floors affected by direct impact of the aircraft and a time-history analysis was generated.

3.3 Response results

Figs.5 through 8 indicate the results of maximum acceleration, displacement, shear force and overturning moment of the analyses carried out. The figures also present the design wind loads and corresponding displacements. Fig.9 shows the time histories of displacements on the 80th and the 110th floors. The maximum acceleration was approximately 3G near the impact floor. The maximum displacement was about 30 cm and 50 cm on the 80th and 110th floors, respectively. Both displacement waveforms shown in Fig.9 consisted of a fundamental mode and a little secondary mode. The overturning moment was about half of that due to the design wind load. The shear force substantially exceeded the design wind load near the impact floor but was less than the ultimate horizontal resistant strength.

4. Structural Damages to the Tower under Impact Load (Local Damage)

4.1 Modeling of the aircraft and the building

LS-DYNA computer program (Halloquist 2001) was used for the analysis. Analytical model was constructed of the 77th through 85th floors of the building that were impacted by the aircraft. Horizontal spandrel beams were assumed to form the boundary conditions at the upper and lower end of the model. The perimeter columns and internal core columns were represented by beam elements obtained by integrating the hollow cross section of a box-shaped column. The clear lengths of the perimeter column and the core were divided into four and three elements, respectively. The four bolts connecting perimeter column units were modeled with constrained weld option. The perimeter spandrel beams were modeled with shell elements of a thickness equivalent to actual thickness. The floor slabs were modeled with concrete shell elements of a thickness approximately 30 cm, in view of the axial stiffness of components of the slab such as concrete, deck plate, and inclined top chord of the floor truss beam. The initial axial force due to gravity load was assumed in each column. For columns and spandrel beams, a failure strain of 10% was assumed. The failure strain of the floor slab was set at 1%. The failure of the bolt connecting perimeter column columns was determined by checking the combination of the axial and shear forces acting at the boundary between the units.

An aircraft, Boeing 767-200ER of length 48.5 m and wing span 47.6 m, was assumed in the model. The total mass at the impact was 142.5t, the sum of the aircraft of 112.5t and the jet fuel 30t (ASCE/FEMA 2002). All structural components of the aircraft were represented by shell elements. The model was constructed so that longitudinal distributions of weights and strengths of the actual aircraft will be represented as accurate as possible, based on data available. The models of the building and the aircraft are shown in Fig.10.
4.2 Analytical conditions

There was no precise available information about the impact location and the attack angle of the aircraft. From the report (ASCE/FEMA 2002), it is assumed that the aircraft flew with its nose tilted 11.5 degrees to the east and 5 degrees downward, and its left wing inclined downward by 35 degrees. The affected areas were estimated based on the damage locations as shown in the Fig.11 (a). The impact velocity of the aircraft at the time of crash was assumed at 590 mph, that is 262 m/sec (ASCE/FEMA 2002). The viscous damping coefficient was assumed to be proportional to the initial stiffness, damping factor 2% for the fundamental mode because damping has a small effect on local damage.

Fig. 5  Maximum acceleration  
Fig. 6  Maximum displacement  
Fig. 7  Maximum shear force  
Fig. 8  Maximum OVTM  
Fig. 9  Displacement response time-history  
Fig. 10  Analytical model of LS-DYNA for local damage estimation
4.3 Analyses of results

An impact analysis using LS-DYNA revealed that 142 perimeter columns and one inner core column were fractured with an effective plastic strain for eight inner core columns exceeding 5%. The nose of fuselage glided through the north wall at a velocity of 94 m/sec in 0.41 seconds after impact. The right engine hit a spandrel beam on the north face and penetrated the building at a velocity of 64 m/sec.

A comparison between the damage to the exterior walls of the analytical model and the actual damage is shown in Fig.12 (ASCE/FEMA 2002). Fig.13 compares the velocity reduction curve of the aircraft with those observed from the video records and analytical results. The analyses revealed that the damage to the area near the boundaries at the upper and lower edge was slightly larger than actual damage. This could be explained by the fact that the stress wave was totally reflected from the boundary surface hence the boundary condition was represented by horizontal rollers in the model. The velocity reduction curves for the tail of the aircraft and of the nose after penetration through the building, and the velocity of the right engine perforating out of the building were in good agreement to the actual observation during the actual incident. Hence, this validated the results of the analytical model.

The damages of inner core columns are shown in Table 2. The damaged areas were concentrated on the east side near the impact area hit by the crash. Only one column (two elements indicated by C in the table) collapsed on the 80th floor. Eight columns (17 elements) suffered an effective plastic strain exceeding 5% (marked by P in the table) on the 79th through 81st floors, which were also assumed unable to carry the gravity loads. The damage of slabs is shown in Table 3. The slabs sustained heavy damage on the 81st floor hit by the fuselage. Twenty elements (an area of approximately 87 m²) were fractured near the perimeter.

5. Redistribution of Vertical Loads after the Impact

5.1 Analytical model

To assess the re-distribution of axial stress in the column after the impact, the entire frame was modeled for gravity load analysis. All the columns and beams on the perimeter of the tubular structure and inner core were represented by general beam elements. The diagonals of the top outrigger truss system were represented by truss elements. The floors were assumed to be rigid in horizontal direction. The analyses were undertaken for two cases under pre and post-impact conditions. In the post-impact model, damaged structural elements were removed, based on the analysis described in Section 4. In either case, vertical load equivalent to the gravity load was applied to the nodes of the analytical model.

5.2 Analytical results

Fig.14 shows the contours of the axial force on the perimeter column before and after the impact. Before the impact, the axial force was almost uniform at the same floor level, though there is a slight uneveness of the contour due to the outrigger truss. After the impact, the axial force from the upper floors was transmitted around the destroyed area. Although the concentration of stresses caused the axial stress of the adjacent column to exceed the yield strength, it is assumed that some surrounding columns jointly carried the axial forces so the average axial forces were less than the yield strength. Fig.15 shows the post-impact axial force of the frame at section A-A (Fig.1) that suffered the failure of many core columns. It is clear that the axial force that should have been carried by columns 1001 through 1005 was transmitted to columns 1006 through 1008 and perimeter columns in section A-A frame owing to the small beams that connect the core columns and the effect of the outrigger trusses in the upper floors. Fig.16 shows the distributions of axial forces on the columns on the 90th, 80th and 70th floors. In the perimeter structure, the axial forces on the remaining columns increased on the south, north and east faces where many columns were damaged. In the core, although the axial forces in the core columns mainly on the north side increased corresponding to the axial force that should have been carried by the damaged columns, the axial stresses of the core columns were less than the yield stress of 250 N/mm². The analysis revealed that the top of WTC2 inclined approximately 21 cm eastward and 12 cm southward after the impact. The inclination angle of the upper portion above the impacted floor was less than 1/500.
Thus, the gravity load carrying capacity of the entire structure after it was damaged by the impact was analyzed. Analytical results show that the axial forces that should have been carried by the damaged perimeter and inner core columns were re-distributed to the remaining columns through the core beams and outrigger truss, and the vertical load carrying capacity was maintained after the impact.

Conclusions

Based on the analyses and stresses conditions resulting from the model it was concluded that:

The entire building did not collapse immediately after the impact as the peak acceleration was 3G, the displacement at the top of the building was 50 cm, and the story shear force and overturning moment were below the ultimate horizontal resistant strength although the shear force and the moment exceeded the wind design force on the upper floors.

The damage to the exterior walls and the velocity reduction curve of the aircraft were in good agreement to the actual phenomena. Although the number of fractured core columns cannot be determined the analytical results show that eight columns at the maximum were fractured. It was revealed that the outrigger trusses helped maintain the gravity load carrying capacity of the entire structure. The floors above the impact dipped approximately 1/500 to the southeast after the impact.

Acknowledgements

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Tsuruta, A. (1970), Steel and Bridge Fabrication Technology of USA+Recent Skyscrapers in USA(1), JSSC Vol.6 No.53
Fig. 11 Comparison of damage of South face (Impact face)

(a) Actual damage (ASCE/FEMA 2002)

(b) Analysis

Fig. 12 Damage of exterior columns by analysis

Fig. 13 Comparison of velocity reduction between observation and analysis

Table 2 Damage of inner core column

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<thead>
<tr>
<th>Floor</th>
<th>1001</th>
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<tr>
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<td>P</td>
<td>P</td>
<td>P</td>
<td>P</td>
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</tr>
</tbody>
</table>

Note) 901-903,1001-1005: column number (refer to Fig.1)
C: effective plastic strain >10%
P: effective plastic strain > 5%

Table 3 Damage of slabs

<table>
<thead>
<tr>
<th>Floor</th>
<th>Collapsed elements / Area</th>
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<td>- / -</td>
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<td>82</td>
<td>- / -</td>
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<tr>
<td>81</td>
<td>20 / 87.90 (m²)</td>
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<tr>
<td>80</td>
<td>6 / 27.71 (m²)</td>
</tr>
<tr>
<td>79</td>
<td>6 / 25.08 (m²)</td>
</tr>
<tr>
<td>78</td>
<td>- / -</td>
</tr>
</tbody>
</table>
Fig. 14  Axial force of exterior columns before and after impact

Fig. 15  Axial force distribution of A-A frame (refer to Fig.1) after impact

(a) 90th floor

100N/mm²
(0.44)

87N/mm²
(0.38)

117N/mm²
(0.51)

111N/mm²
(0.48)

152/mm²
(0.66)

183N/mm²
(0.80)

128N/mm²
(0.56)

122N/mm²
(0.53)

105N/mm²
(0.46)

125N/mm²
(0.54)

133N/mm²
(0.58)

( ) : ratio of axial stress to yield stress

Fig. 16  Redistribution of axial force of inner core and perimeter columns