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Appendix B
INTERIM REPORT ON DEVELOPMENT OF STRUCTURAL DATABASES AND REFERENCE MODELS FOR THE WTC TOWERS

B.1 INTRODUCTION

The objectives of this project are to (1) develop structural databases for the primary structural components of the World Trade Center towers, (WTC 1 and WTC 2), (2) use the databases to develop reference structural analysis models that capture the intended behavior of each of the two towers, and (3) perform linear, static structural analyses to establish the baseline performance of each of the two towers under design gravity and wind loads. This appendix focuses on the tasks related to the first two objectives. The appendix reports on the work conducted by the firm of Leslie E. Robertson Associates (LERA), the firm responsible for the original structural engineering of the WTC towers, for the development of the structural databases and reference models. It also outlines the comprehensive review process for the structural databases and reference models that includes the rigorous in-house NIST review and third-party review by the firm of Skidmore, Owings, and Merrill (SOM).

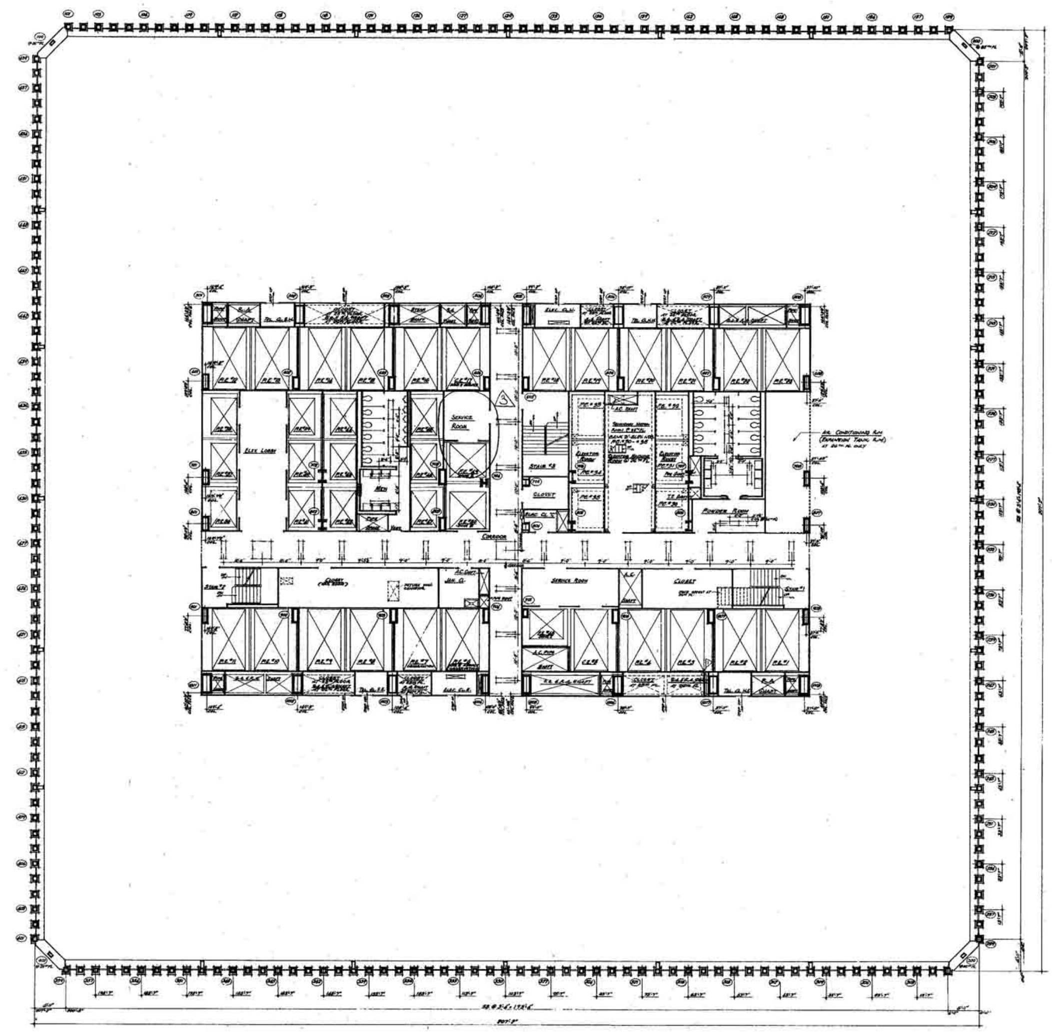
Section B.1 presents an introduction and a brief description of the structural system of the towers. Section B.2 presents an outline and methodology used for the development of the structural databases for both towers from the original computer printouts of the structural documents, along with the relational databases that are used for the development of the reference structural models. Section B.3 presents the development of the reference structural analysis models for WTC 1 and WTC 2, including global tower models, typical floor models, and parametric studies needed for the development of the global models. Finally, Section B.4 outlines the in-house and third-party review process for the structural databases and reference models.

B.1.1 Description of WTC Structural System

Global Structural System

WTC 1 and WTC 2 each consisted of a 110-story above grade structure and 6-story below grade structure. The buildings, which were each approximately 207 ft by 207 ft square in plan and with story heights of typically 12 ft, rose to heights of 1,368 ft (WTC 1) and 1,362 ft (WTC 2) above ground. The exterior walls of the towers supported gravity loads and all lateral loads, and were constructed of steel closely spaced, built-up columns and deep spandrels. The core contained columns that supported the remainder of the gravity loads of the towers. The core area was approximately 135 ft by 87 ft in plan (refer to Fig. B–1). The distances between the rectangular core and the square exterior wall were approximately 36 ft and 60 ft. The areas outside of the core were free of columns and the floors were supported by truss-framing in the tenant areas and beam-framing in the mechanical rooms and other areas.

The primary structural systems for the towers included exterior columns, spandrel beams, and bracing in the basement floors, core columns, core bracing at the mechanical floors, core bracing at the main lobby atrium levels, hat trusses, and the floor systems.



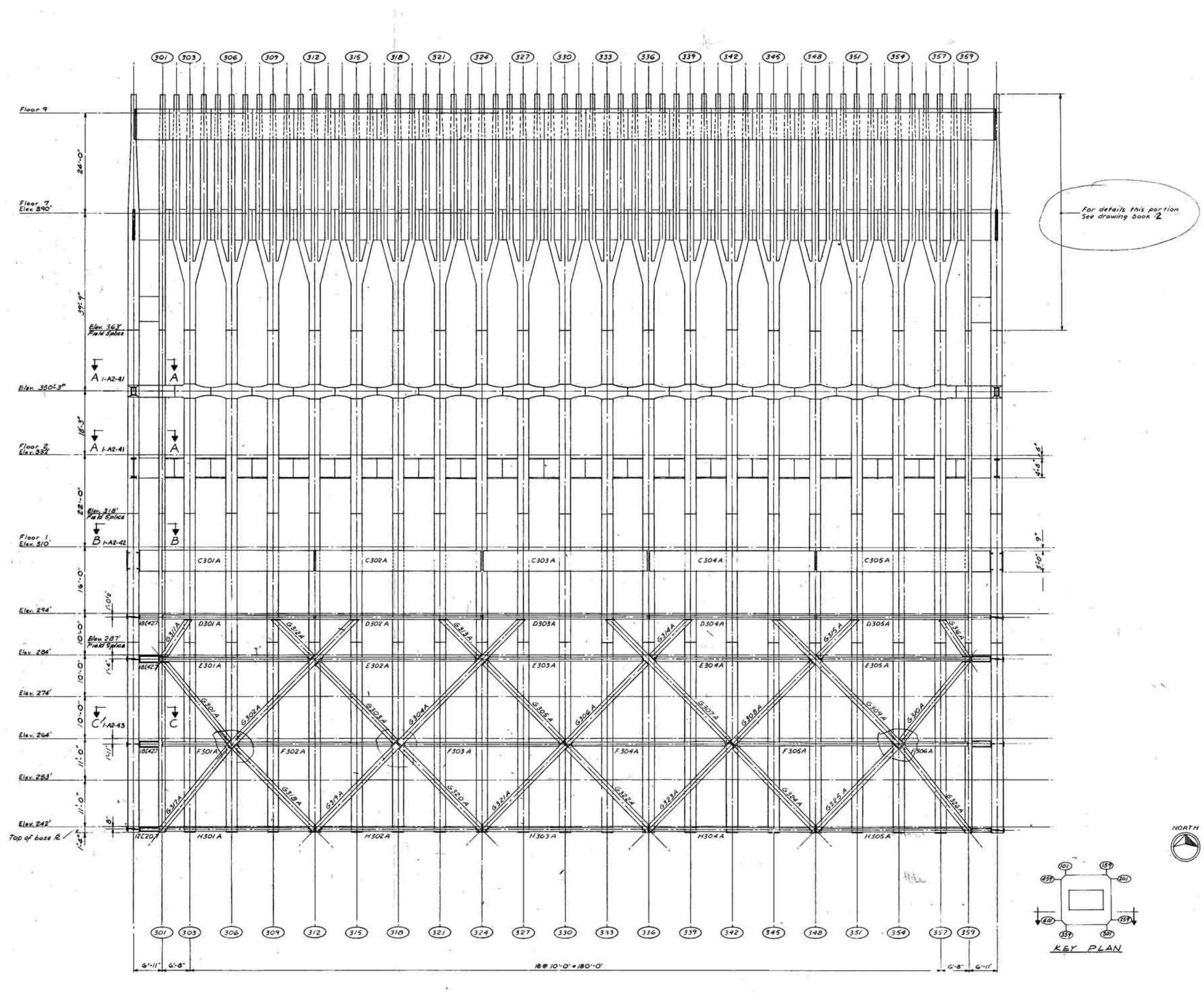
Drawing used with permission from PANYNJ.

Figure B–1. Typical WTC tower architectural floor plan (floor 26, WTC 2).

The exterior wall columns from the foundation level up to Elevation 363 ft were spaced 10 ft 0 in. on center, built-up of steels plates, and connected by spandrels. Bracing existed in the plane of the exterior wall between the Concourse Level and the foundation, (see Fig. B–2). Between Elevation 363 ft and floor 7, the single exterior wall columns spaced 10 ft 0 in. on center transitioned to three columns spaced at 3 ft 4 in. on center as shown in Fig. B–2 (see also Fig. B–19).

The exterior wall columns above floor 7 that were spaced 3 ft 4 in. on center, were built-up of steel plates, and were connected to each other by spandrel plates, typically 52 in. deep. The exterior columns and spandrels were pre-assembled into exterior wall panels, typically 3-columns wide by 3-stories tall (refer to Fig. B–22).

The core columns were typically built-up box members at the lower floors and transitioned into rolled structural steel shapes at the upper floors. The core columns were typically spliced at 3-story intervals at

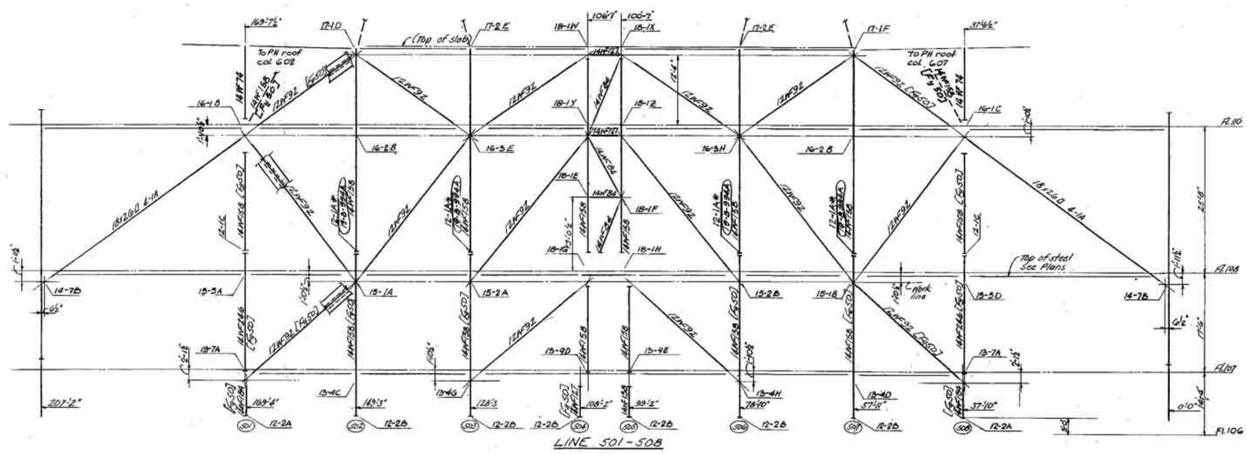


Drawing used with permission from PANYNJ.

Figure B–2. Typical WTC exterior wall, foundation to floor 9.

3 ft above floor level. Diagonal bracing of the core columns existed at the lobby atrium levels, the mechanical levels, and in the area of the hat truss.

At the top of each tower, hat trusses interconnected the core columns with the exterior wall panels and provided a base for the antennae. The vertical members of the hat trusses were wide flange core columns. The diagonals were primarily wide flange rolled sections with the exception of the end diagonals interconnecting the core to the exterior walls which were built-up box sections. The majority of the horizontal members in the hat truss system were wide flange and built-up box section floor beams. The members of the hat trusses were shown in the SA/B-400 series elevations (refer to Fig. B–3).



Drawing used with permission from PANYNJ.

Figure B–3. Typical WTC tower hat truss elevation (Drawing SA 401).

Floor Structural System

In the typical WTC tower floor plan, the area inside the core was framed with rolled structural steel shapes acting compositely with formed concrete slabs. The area outside the core was framed in either trusses (typical on tenant floors) or in rolled structural steel shapes (typical on mechanical floors).

Truss-Framed Floors—The majority of the floors of the WTC towers were tenant floors where the areas outside of the core were constructed of steel trusses acting compositely with concrete slabs cast over metal deck. The trusses consisted of double angle top and bottom chords with round bar webs and were designed to act compositely with the concrete slab. Composite action was achieved by the shear connection provided by the web bar extending above the top chord and into the slab. Two trusses were placed at every other exterior column line, resulting in a 6 ft 8 in. spacing between truss pairs. The typical floor consisted of three truss zones: a long span zone, a short span zone, and a two-way zone, (see Fig. B–4).

The floor trusses were pre-assembled into floor panels as defined in the contract drawings. The span of the trusses was about 36 ft in the short direction and 60 ft in the long direction. The floor panels included primary trusses, bridging trusses, deck support angles, metal deck, and strap anchors, all of which were defined by the contract drawings and specifications.

The floor truss panel types are indicated in the structural plans (see Fig. B–5) and the plans refer in turn to Drawing Book 7 for information regarding the components of the floor truss panels and to Drawing Book D for damper information. Drawing Book 7 provided panel by panel layout plans and elevations of each referenced truss. The section through a floor panel after the concrete was placed is illustrated in Fig. B–6.

Beam-Framed Floors—The typical locations of the beam-framed floors were the mechanical floors, the mechanical mezzanines, and the floors above the mezzanines (e.g., floors 41, 42, and 43). These floors were constructed using rolled structural steel shapes. The beam framing for the typical floor system was W27 beams in the long span region and W16 beams in the short span region. Typically, beam spacing was 6 ft 8 in. The steel beams acted compositely with the normal weight concrete slab on metal deck.

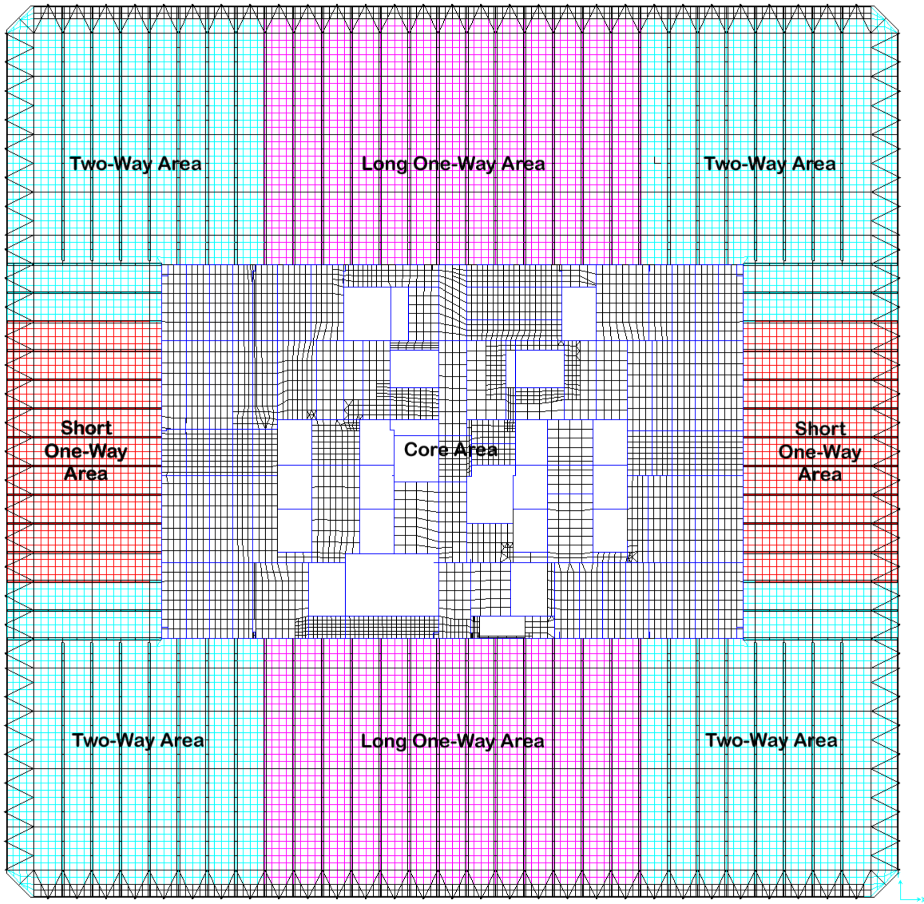


Figure B–4. Typical WTC floor truss framing zones.

The deck spanned in the direction of the primary beams and was supported typically at 6 ft 8 in. intervals by a 4C5.4 deck support channel. A 2 in. concrete topping slab was placed on top of the structural slab. The core area was framed similar to the core of the truss-framed floors, but the steel beams were typically larger, and the concrete slab was 6 in. deep. The beam-framed floors above the mechanical mezzanine had a 7 3/4 in. normal weight concrete slab on 1 1/2 in. metal deck, while the core slab was 8 in. normal weight concrete.

Beam-framing was added to truss-framed floors at levels which supported escalators or stairs in the areas outside of the core. The escalator floors occurred typically in the two levels directly above the mechanical rooms.

B.2 DEVELOPMENT OF STRUCTURAL DATABASES FOR THE TOWERS

This section outlines the development of the electronic databases for the major structural components of the WTC towers from original computer printouts of the structural documents. The structural databases are used to develop the reference structural models of the towers as outlined in Section B.3. Included in this section are an overview of the WTC towers' structural design documents, a description of the structural database contents, methodology for the development of the database, and a description of the relational database.

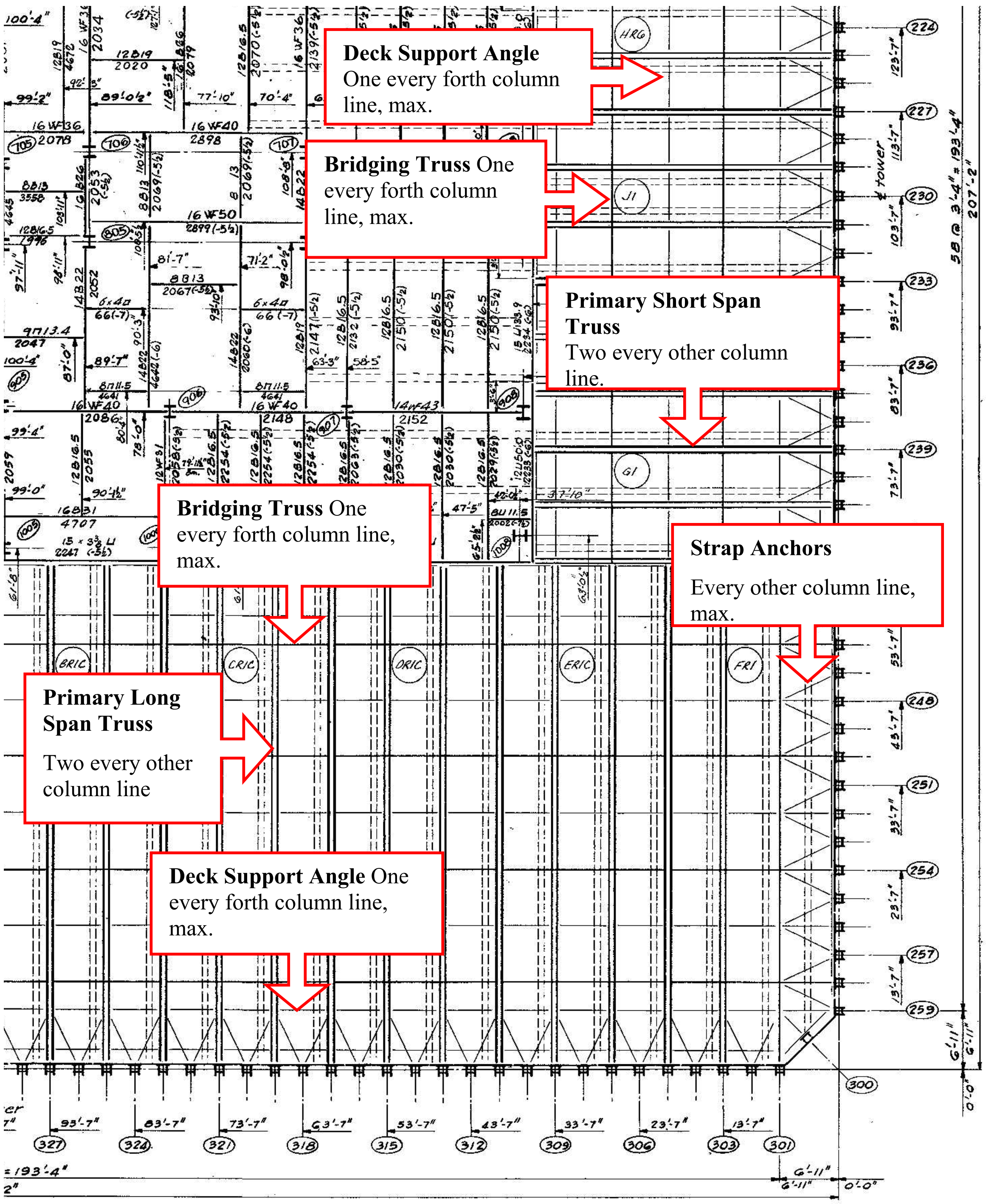


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Figure B–5. Part plan of floor 96 of WTC 1 (Drawing SA-104), components of typical truss framing system.

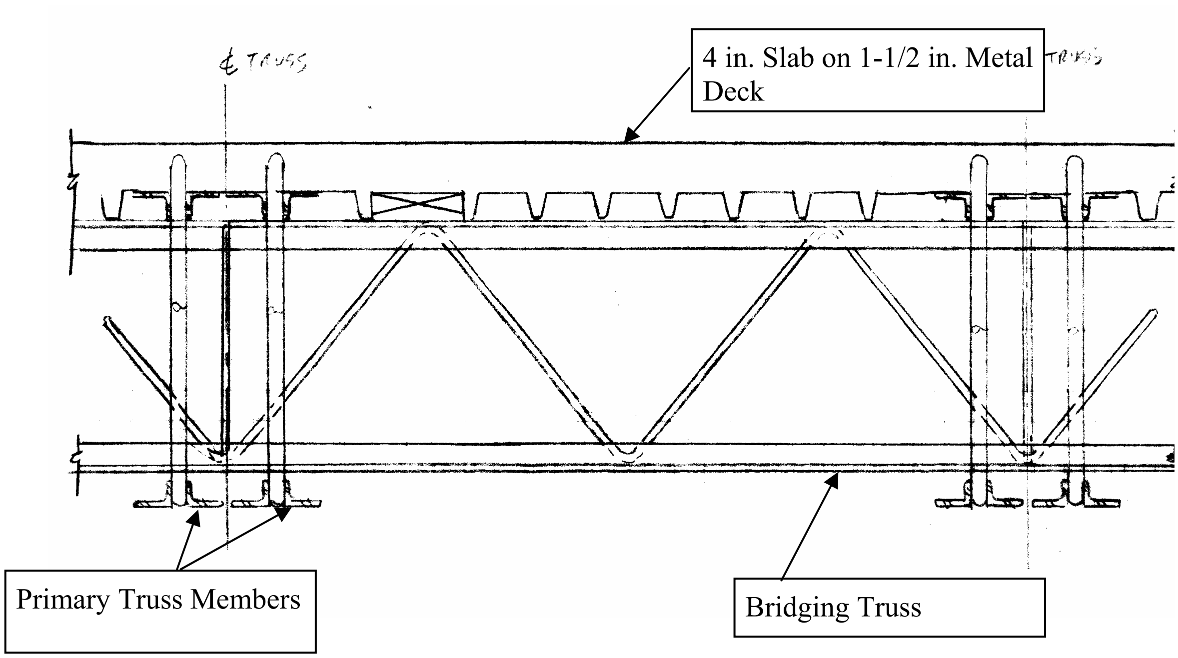

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Figure B–6. Part section typical truss floor panel.

B.2.1 Description of WTC Structural Documents

The WTC structural drawings were issued in two main formats: large-size sheets containing plan and elevation information and smaller book-sized drawings containing details and tabulated information. Throughout the WTC drawings, Tower A or WTCA denotes WTC 1 (north tower) and Tower B or WTCB denotes WTC 2 (south tower). The large size drawings always make reference to the structural drawing books through their notes, sections, and detail references. The structural drawing books for WTC 1 and WTC 2 include the following:

- Book 1 contains exterior wall information to elevation 363 ft. (Dates: 02/1967 to 12/1968, Approx. 213 pages).
- Book 2 contains exterior wall information elevation 363 ft to floor 9. (Dates: 04/1967 to 12/1967, Approx. 62 pages).
- Book 3 contains core column information. (Dates: 03/1967 to 09/1969, Approx. 137 pages).
- Book 4 contains exterior wall information floor 9 to floor 110. (Dates: 04/1967 to 10/1972, Approx. 1,080 pages).
- Book 5 contains the beam schedule. (Dates: 05/1967 to 08/1969, Approx. 292 pages).
- Book 6 contains connection details and core bracing. (Dates: 08/1967 to 05/1969, Approx. 1,060 pages).

- Book 7 contains truss floor panel information. (Dates: 10/1967 to 07/1969, Approx. 345 pages).
- Book 8 contains concrete notes and details. (Dates: 03/1968 to 07/1974, Approx. 926 pages).
- Book 9 contains roof area column splice details. (Dates: 05/1970 to 04/1971, Approx. 440 pages).
- Book 18 contains strap anchor and core truss seat information. (Dates: 10/1968 to 11/1969, Approx. 219 pages).
- Book 19 contains revisions after fabrication. (Dates: 08/1968 to 05/1975, Approx. 374 pages).
- Book 20 contains structural steel details. (Dates: 07/1968 to 03/1971, Approx. 41 pages).
- Book D contains damper details. (Dates: 03/1969 to 09/1971, Approx. 43 pages).

The remaining number books (Books 10, 11, 12, and 13) contain information about the sub-grade structure. Books 14, 15, 16, and 17 were never used.

Until fabrication was begun, the above drawings and drawing books (with the exception of Book 19) for the project were modified in keeping with the requests for changes by contractor(s) and early tenant modifications. The drawings were modified up until such time as the fabrication of elements commenced. At that time, Book 19 was introduced. It contained the information regarding 'revisions after fabrication'.

LERA believes that the original structural drawings represent significantly accurate 'as-built' drawings for the towers. As tenant modification requests became large in scope, they became separate projects (e.g., the Fiduciary Trust Vault Project, see Section B.2.4). Tenant structural modifications designed by LERA were then documented in a single book of quarter-size plans referred to as the 'WTC Tenant Structural Modifications Book'. Later tenant modifications were mostly archived on a job-by-job basis without a central accounting for all the changes. NIST has in its possession complete copies of all the drawings, drawing books, and modifications to the towers performed by LERA. In some instances modifications were made by the Port Authority of New York and New Jersey (PANYNJ) Engineering, such as additions to the mechanical levels. In other instances, tenant modifications were performed by other engineers. For these instances, LERA does not have record of the work completed. According to the PANYNJ, no record of structural work could be found so far for the additions to the mechanical floors made by the PANYNJ Engineering, and it is likely that they were lost with the collapse of the towers. Modifications made by other engineering firms include openings or closings of floor slabs and local reinforcement of floor segments to accommodate new loads. For these modifications, NIST has access to the documents related to the work.

The few modifications made by LERA to the components compiled in the WTC structural databases that will have an effect the global behavior of the towers are listed in Table B–1.

Table B–1. Modifications to members of the WTC database (WTC-DB).

Item	Description	Tower	Element	Floor	Element Effected	WTC-DB Modified	Archived
1	Core column reinforcing	WTC 1 and WTC 2	Numerous	98–106	Core columns	Book 3	Book 19
2	Fiduciary Bank Vault	WTC 2	Col. 508B and Col. 1008B	45–97	Core columns	Book 3	LERA P209
3	Bombing of 26 February 1993 repair	WTC 1	Col. 324, bracing G313A and G304A	B-2 level	Perimeter column and bracing	NA	LERA P1003118
4	EXCO stair	WTC 1	Col. 901A	26	Core column	NA	LERA P1003249

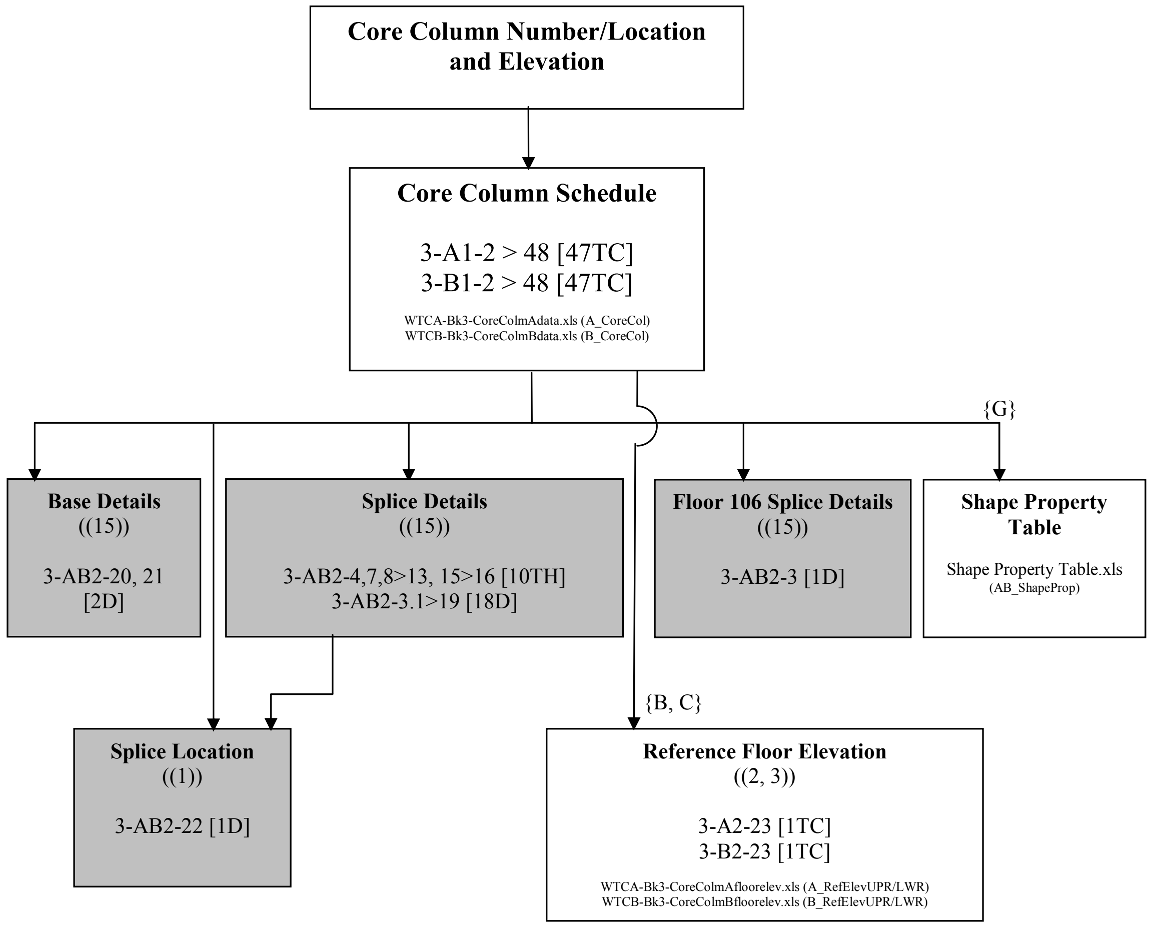
B.2.2 Overview of the WTC Structural Database (WTC-DB)

The original WTC design documents used the concept of limiting the need for repetition in documenting the data shared between different elements with similar characteristics. The drawing book schedules refer to subsequent tables for information common to several lines of the same schedule. In an effort to minimize the amount of repeated information and thereby the data checking of the digital WTC-DB, the drawing book data within the databases created for this project were linked in a similar manner. In order to accurately follow the original flow of the drawing book links, flowcharts of the drawing books to be digitized were developed for this project. These flowcharts were used to organize the links of the digitized data within the relational database. An example of such flowcharts for Drawing Book 3 (core columns) is illustrated in Fig. B–7.

The WTC-DB contains the computer and hand-tabulated data for the major structural components from the original Drawing Books 1 through 5, including exterior walls, core columns, and beam schedule for the towers. Where information from Drawing Books 1 through 5 was modified by Drawing Book 19 and would affect the towers' modeling, the information is included in the database. In addition, some information from Drawing Books 6 (core bracing schedule) and 9 (beams in the hat truss region) has been included in the database files as it was utilized in the finite element modeling of the towers.

The drawing book tables are first digitized and stored in Microsoft Excel format files. The Excel files include several worksheets that describe the evolution of the data from the drawing book to the final database format, as well as additional information and notes for interpreting the data.

The WTC relational database links the Excel files and allows users to view and select data through query commands. The primary benefit of the relational database format is the ability to programmatically query the database for data required in assembling the structural models of the towers. The query routine allows multiple users the ability to review, extract, and export the basic data in any required form. The data can be manipulated using Structured Query Language (SQL) according to the desired output, for example the structure of the user's finite element model input file.



NOTES:
1. 4-AB-* Denotes Drawing Book 4, Tower A , B or AB, and page number
2. [*TC] Number of pages and type, where page types include:
   TC - Computer generated tables; TH - Hand written tables; D - Diagrams
3. *.xls Excel spreadsheet file name; (AB_***) database heading
4. ((*)) Reference note key from original Drawing Book information table.
5. {*} Relational database link (i.e. Excel column number) from previous *.xls file.
6. Figures of columns and panels are shown from inside of building looking out, unless otherwise noted.

Figure B–7. Drawing Book 3 flowchart: WTC 1 and WTC 2 core columns, foundation to floor 106.

B.2.3 Methodology for the WTC-DB Development

Data Entry

The tabulated portions of WTC Drawing Books 1, 2, 3, 4, 5, 6, and 9 were first scanned and stored in TIFF image format files. The image files containing the tabulated information were then opened in an Optical Character Recognition (OCR) program that converted the information into a text file. The OCR program was modified to allow for the filtration of unnecessary characters during the documents conversion process. In other words, the user could direct the program to block specific characters that are not on the actual page. As an example, if after reviewing a table, one recognizes that it uniformly contains numbers and only the characters "A, B, -", and "/" then the remaining characters can be frozen out by the software. This reduced the misinterpretation, as an example, of a 'Z' for a '2', or an 'O' for a 'zero'.

The raw text file was then opened in a word processing program, where it was compared with the original hardcopy drawing books. As needed, data columns were adjusted, and obvious errors were individually corrected. The 'cleaned' text file was then imported into a Microsoft Excel spreadsheet with column headings and proper alignment. When importing into Excel, "text" was the cell format used for handling of the scanned information to avoid the misinterpretation of fractions as dates (e.g., 3/8 as March 8). An Excel macro was written at this stage to convert the text fractions into number fractions. The final product of this stage was an Excel file that contains the information from the drawing book table.

Quality Control

Checking began during the OCR data entry process, where the files being entered and the OCR software interpretation were viewed simultaneously. This was considered a first check. Once the Excel file was complete, the file entered the 'second check' process.

The 'Second Check'—An engineer not involved during the OCR process performed a second, sample checking of the database in a random but methodical manner. For approximately once in four pages, every cell of data in the page was compared with the original drawing books. Discrepancies of the files were then either re-entered using OCR or were individually corrected to agree with the original books.

The 'Cross-Check Rectify' Check—After completing the 'second check,' the files were compared with the database provided by a consultant for the leaseholder of the WTC towers as part of an insurance litigation concerning the towers (provided to LERA as Government Furnished Information [GFI]) using a cross-check macro formula worksheet. Once compared, conflicting information appeared in a yellow highlighted cell displaying both sets of compared information in the "Calculation" macro formula worksheet. The cell was then reviewed and confirmed with the WTC drawing books. If errors were from the developed worksheet, data was rectified, and the yellow highlight in the 'Calculation' worksheet was then removed by comparing the files again. If errors were from the GFI worksheet, raw GFI data was not modified, but the cell was highlighted in blue to note that it has been reviewed. The files were then compared again, and the cell color in the 'Calculation' worksheet changed to blue. The process was repeated to remove all the yellow cells so that only blue highlighted cells remain. The worksheet 'ComparisonORIGINAL' was retained for the record of the original comparison, and the updated worksheet 'ComparisonFINAL' was retained for the record of the final comparison.

Final Review**—**Finally, the files were reviewed for completeness, formatting, and data units. A final check was made to find any numbers that may have been input as text letters. Following this review, the worksheet was used to develop the member section properties.

Cross Section Property Calculations

The next step was to calculate the cross section properties for the members included in the database. The section properties calculated included cross sectional area (A), moment of inertia (I), section modulus (S), plastic section modulus (Z), radius of gyration (r), and torsional constant (J) for both the major and minor axes (where applicable). The *Section Designer* function of SAP2000 Version 8 was used to calculate the cross section properties since it enables the program to perform more precise code checks, as the dimensions of each plate element that is part of the section would be input into the finite element model.

The current rolled shape database in SAP2000 represents the modern day rolling practices. The rolled shapes used in the construction of the WTC towers were from a different era and thus, had different properties in comparison to present day shapes. Therefore, a rolled shape database consistent with the time of construction was developed in this project. See Section B.2.5 for further discussion about the rolled shape database.

Relational Database Development

As discussed earlier, the original WTC drawing books were designed to avoid repeating identical information. The drawing book schedules, therefore, refer to other tables for information common to several lines of the same schedule. In keeping with the nature of the original drawing books and to minimize the data in the digital WTC-DB, the drawing book and section property data were linked using Microsoft Access.

The assembly of the relational database began with the mapping of the original WTC drawing book into flowcharts (see, e.g., Fig. B–7). The digitized drawing book data with the corresponding cross sectional member properties from the Excel-format files were then imported into the Microsoft Access database program and partitioned into tables. The tables were then joined using the links cataloged in the flowcharts. These tables were developed to provide the input files for the finite element modeling of the towers as illustrated in Section B.3.1.

B.2.4 Modifications to Database Elements

The majority of the original members and elements defined within the WTC-DB could be fully defined by the original data in the drawing books. As outlined in Table B–1, however, some modifications were made that are described in the following sections. Of the items outlined in Table B–1, items 1 and 2 have been included within the database.

Core Column Reinforcing at Floors 98 to 106

A number of core columns in both WTC 1 and WTC 2 were reinforced at floors 98 to 106. Book 19, pages 19–AB–974.1 through 4, shows that core columns 501, 508, 703, 803, 904, 1002, 1006, and 1007 from floors 98 to 106 in both towers were reinforced with steel plates. Three methods were used to attach the reinforcing plates to the wide flange columns: (1) the plates were welded to the flanges; (2) the plates were welded to the webs; and (3) the plates, which were parallel to the web, were welded to the flange ends. The plate information (width, thickness, length, and yield strength) was incorporated into the database tables of Book 3. Since the plates varied from floor to floor, the original column (defined over a three-story height) was split into typically three floor-by-floor sections and the designation of the column was appended to include either U(upper), M(middle), or L(lower) designation (refer to Fig. B–8). For floors 104 and 106, the columns are two-story columns. Hence, the columns at these floors had only U and L designations. The section property calculations included the contributions of the reinforcing plate at each level. For the built-up section property data, the reinforcing plate was considered to be applied to the column for its floor-to-floor height.

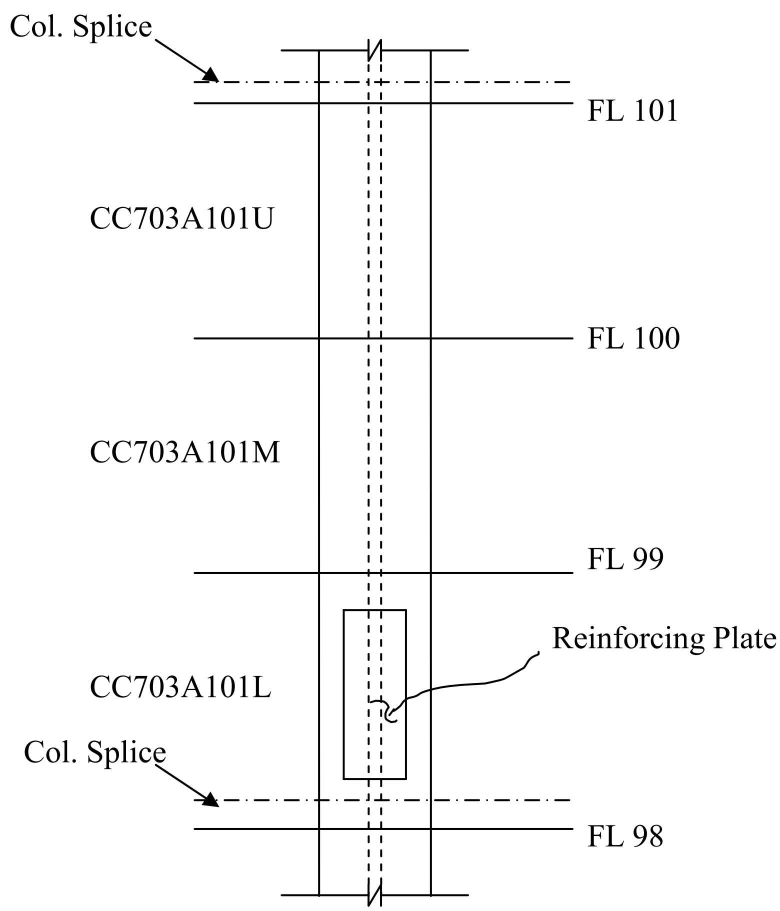


Figure B–8. Core column reinforcement.

Core Column Reinforcing Due to Construction of Fiduciary Trust Vault

The Fiduciary Trust Company added a concrete vault at floor 97 of WTC 2, which required reinforcing two corner core columns at the north end of the core. This work is included in the WTC Towers A and B Structural Renovation Drawings Reference Manual. The Fiduciary Trust Structural Drawing 765–S–A–4 shows that WTC 2 core columns 508 and 1008 were reinforced with steel plates from floors 45 to 97. The reinforcement consisted of plates welded to the flanges of the built-up box columns (floors 45 to 83) and the flanges of the rolled shape columns (floors 83 to 97). These reinforcing plate modifications and the reinforcing plates yield strength, F_Y, were added to the original Book 3 data contained in the WTC-DB. The database included plates that extended long enough such that they substantially affect the member properties of the column, e.g., the added plates increase the capacity of the columns. Where the plates appeared to reinforce only the column splice, they were not included in the database.

The reinforcing plate data were tabulated and incorporated into the database in the same manner as the plates discussed in the previous section, except that a second length designation was added to differentiate the length of the plates on the north and south faces of the columns, i.e., the column designation "LN" refers to the length of the plates on the north face of the column. Again, when calculating the built-up column properties, the plate was assumed to be continuous along the floor-to-floor height of the column. When the length of the reinforcing plate shown in the drawing was greater than the floor height, the plate was attributed to the two column segments. Where the plate extended over the entire height of the segment, the length was tabulated as the height of the column segment. The remaining length of plate was attributed to the other column segment.

Repair Due to the Bombing of February 26, 1993

The 1993 bombing resulted in structural damage to WTC 1, centered at exterior column 324 (south wall), B-2 level. The face of the column towards the explosion was slightly bowed, and the splice in the column developed a hairline crack. The column was reinforced locally to account for the loss of steel area. The bracing on either side was replaced with equivalent sections and attached in a similar manner as the originals. No modification to the WTC-DB has been made for this repair.

Tenant Alteration for an Interoffice Stair

A tenant alteration was provided for an interoffice stair between floors 25 and 26 in WTC 1. This work, adjacent to core column 901A, was performed by an engineering firm (other than LERA) and unknowingly resulted in the loss of a core column bracing strap (refer to Fig. B–9), leaving the column unbraced about its minor axis for two stories. The PANYNJ alerted LERA to the issue and asked LERA to review. The situation was reviewed by LERA, and the column stability was found to be adequate. No modification to the WTC-DB has been made for this modification. The effect of removing the strap is accounted for in the global model of WTC 1, see Section B.3.1.

Drawing Book Data Discrepancies

In the original WTC drawing book data, the following discrepancies were discovered by LERA:

- Book 1 page 1–B–15. For member number G311A, the inch portion of the length is listed as 3-1/18. Based on the comparison to similar bracing types in the area, this dimension was modified to be 3-1/8 in. in the WTC-DB.

- Book 3 page 3–A1–10. For core column 601A between floors 86 to 89 and 89 to 92, the column type is listed as 213. Type 213 is a column type which by definition has reinforcing plates, but for this location, no plate data was provided in the schedule. This, in combination with comparisons to similar columns in plan, led to modifying the column type to 111. This also applies to column 601B, page 3–B1–10 between floors 86 to 89 and 89 to 92.

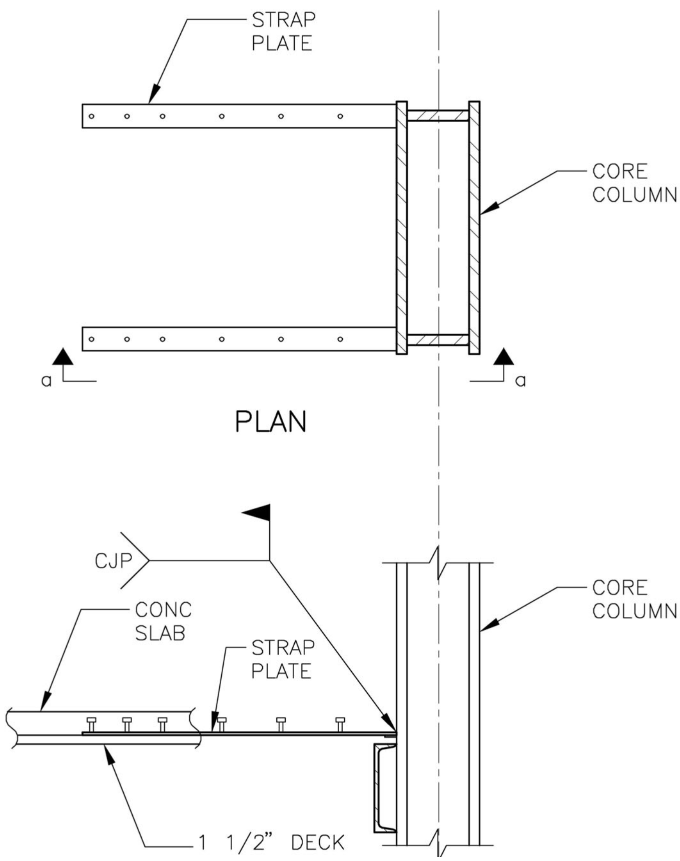


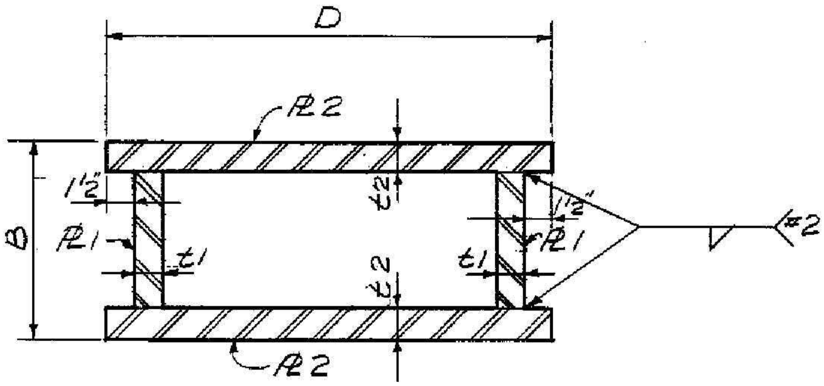
Figure B–9. Column section at original column strap detail (taken from drawing book 18, page 18–AB2–12).

- Book 3 page 3–B1–48. For column 1008B between floors 63 to 66, the yield strength, F_Y, is listed as 6 ksi in the table. Based on the yield strength of the columns above and below these floors, the yield strength was modified to be 36 ksi. For the same column number and floor segments, the lower splice detail number is listed as " 01G." Based on the lower splice detail number of the columns above and below these floors, the number was modified to be "301G."

- Book 3 page 3–B1–9. For core column 508B between floors 21 to 24, the length of plate 1, *W1*, is tabulated as 11.25 in. However, length *B* for this column is 22 in. and thickness *t2* is 5.5 in. *W1* equals *B* minus two times *t2* (see Fig. B–10). Hence, assuming *t2* was listed correctly in the table, *W1* was modified to be 11 in.

- Book 1 page 1–B–23 and 1–B2–19. The details for column types 1024, 1025, 5024, and 6025 listed in the tables are not explicitly shown in the drawing book. For these members, column shapes are assumed to be as shown in the typical details in page 1–B–19 for the 1000 series columns, 1–B–24 for the 5000 series column, and the 1–B–27 for the 6000 series column.

- Book 3 page 3–AB2–6. The column type 216 does not appear to be assigned to any member in the drawing book.



Drawing used with permission from PANYNJ.

Figure B–10. Core column series 300.

B.2.5 Section Property Calculations

SAP2000 *Section Designer* was typically used to calculate section properties for built-up sections. The sections were "built-up" within SAP2000 by defining plate dimensions and offsets from 0-0 location. Section orientations were defined with the X-X axis horizontal to the bottom of the original drawing book page as the detail is shown in the drawing book.

During the process of calculating properties there was an exception to this orientation rule. Core column members CC1007A104L, CC1002A104L, CC703A106L, CC1007B104L, CC1002B104L, and CC703B106L consist of a wide flange shape and web reinforcing plates. These members were input into SAP2000 rotated 90 degrees from the orientation shown in the details to utilize the default orientation of the wide flange section in *Section Designer*. Once the properties were calculated, the sections were placed in the WTC-DB following the orientation of the detail (i.e., the axis was shifted back 90 degrees).

When rolled shapes were used to create built-up sections, the rolled shapes database developed for this project was used to build the sections in SAP *Section Designer* as explained later in this section. The 200 series core columns (wide flange rolled columns reinforced with plates) are examples of members whose properties were calculated in this manner.

Member Designations

For member section property calculations and assembly of the finite element models, the members were named using the following general member designations. The member designations are listed in the Microsoft Excel files.

First character:

- Book 1—below tree–B
- Book 2—exterior wall tree–T
- Book 3—core columns–C

- Book 4—exterior columns and spandrels–E

Second character:

- C—column
- S—spandrel and below grade exterior wall spandrel, strut, or bracing

Third to fifth character: (third to sixth character for 4 digit column e.g., 1004)

- Column number

Sixth character:

- A—WTC 1
- B—WTC 2

Seventh character and above:

- Upper splice level—for core columns
- U(upper), M(middle), or L(lower)—column segment where reinforcing plates are added
- T or B—top or bottom of nonprismatic columns
- Detail letter (lowercase)—(where more than one section is calculated)
- F or C—face or center of nonprismatic spandrel
- Elevation—below tree spandrel elevations

Column Member Multiple Section Property Calculation

In the database, the following three types of column members had different cross sections along the length of the members:

- Exterior wall tree at level C in Drawing Book 2 (two different cross sections)
- Exterior wall tree at level E in Drawing Book 2 (three different cross sections)
- Exterior column type 300 (floor 9 to 106) in Drawing Book 4 (two different cross sections)

For these three member types, the section properties of the different cross sections were calculated and listed in the database tables. In an effort to minimize repeated information, the raw input data for all sections were only shown in the rows that correspond to the first cross section. For the second and third (if any) cross sections, the calculated data followed in the rows below. The constant raw data such as the column number were not repeated in these rows of the table, and thus the corresponding cells were left blank. Since the column number was used as a link for the development of the relational database, only the row containing the raw input data and the first cross section properties was returned in a query, and thus, the user must refer back to the Microsoft Access 'Tables' for the remaining section property information. The section names of the different cross sections along the member length were distinguished by the last one to two characters, which identified the cross sections where the section properties were calculated.

For example, exterior column EC339 (mechanical floors) tapers over a portion of the length of the member (refer to Fig. B–11). The section properties above and below the spandrel were calculated. The column section above the spandrel was called EC339, while the column section below the spandrel was called EC339cc. The suffix 'cc' denoted the section below the spandrel. Note that the raw dimensional data for EC339cc were not shown in the table, as the information was the same as for EC339.

Spandrel Member Multiple Section Property Calculation

In the database, the exterior columns below elevation 363 ft in column series 5000, 6000, and 7000 in Drawing Book 1 had corresponding spandrels shown in the details in Book 1. There were two types of spandrels for these members, tapered built-up box shapes and built-up I shapes. For the tapered built-up box shapes, the section properties of the different cross sections were calculated and listed in the database tables. The data were listed in the database files as described for columns with multiple cross sections. The section names of the different spandrel cross sections along the member length were distinguished by the last three to four characters, which identified the cross sections where the section properties were calculated.

For these exterior columns, there are spandrels at two elevations, 332 ft and 350 ft. At elevation 350 ft, the spandrels tapered, and as a result two cross section properties were calculated. The first section was at the face of the exterior column, and the corresponding section name had a Suffix F (face). The second section was at the center of the spandrel in between two exterior columns, and the corresponding section name had a Suffix C (center). The elevations and locations of the cross sections of the spandrels were shown in the figures in the "Cross Section" worksheets in the database Excel files.

For example, four different section properties were calculated for exterior column 6009 in WTC 1. The first section was the exterior column itself, and the section name was BC6009A. The other three sections, BS6009AB332, BS6009AT350C, and BS6009AT350F were for the spandrel sections. The suffix B332 in BS6009AB332 denoted the bottom spandrel at elevation 332 ft. Suffixes T350C and T350F in BS6009AT350C and BS6009AT350F, respectively denoted the top spandrel at elevation 350 ft, and the "C" or "F" identified the locations where the section properties were calculated, see Fig. B–12.

Section Property Calculation Comparisons

For all the members whose section properties were included in the GFI database, the cross sectional properties in the GFI data were compared with the data contained within the WTC-DB. Most section property results compared with good accuracy between GFI and the WTC-DB (within 1 percent). It was found that results from the calculations of the torsional constant, *J*, however, did vary. LERA in-house

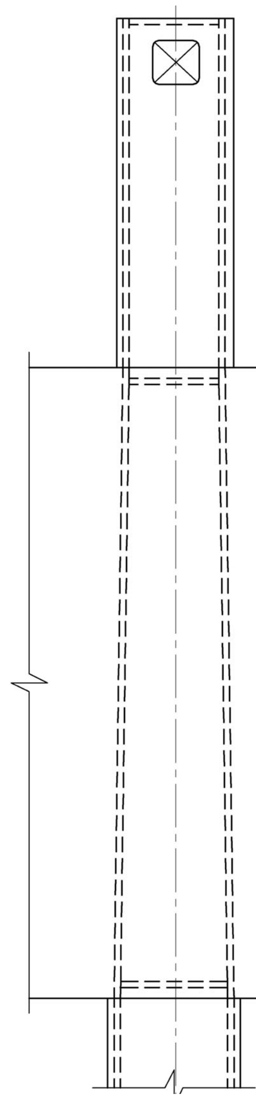


Figure B–11. Exterior column type 300, floor 9 to floor 106 (taken from drawing book 4, page 4–AB2–18).

programs were then used to confirm the accuracy of the *J* calculation. For core columns in WTC 1, SAP2000 generated values used in the WTC-DB were on average 8 percent larger than *J* values calculated using a LERA in-house program, while the results provided by the GFI database were on average 13 percent greater than LERA in-house program *J* calculations.

It was found that for box sections, *J* values calculated by the above equation matched the *J* values given by SAP *Tube Section*. However, for the same tube section, the *J* values given by SAP *Section Designer* were greater than *J* given by SAP *Tube Section*, even while all other properties were equivalent. According to Computers and Structures, Inc., the developer of SAP2000; the *J* values given by SAP *Section Designer* are more accurate as SAP *Section Designer* uses a finite element method to calculate the *J* values while an approximate equation is used in SAP *Tube Section*.

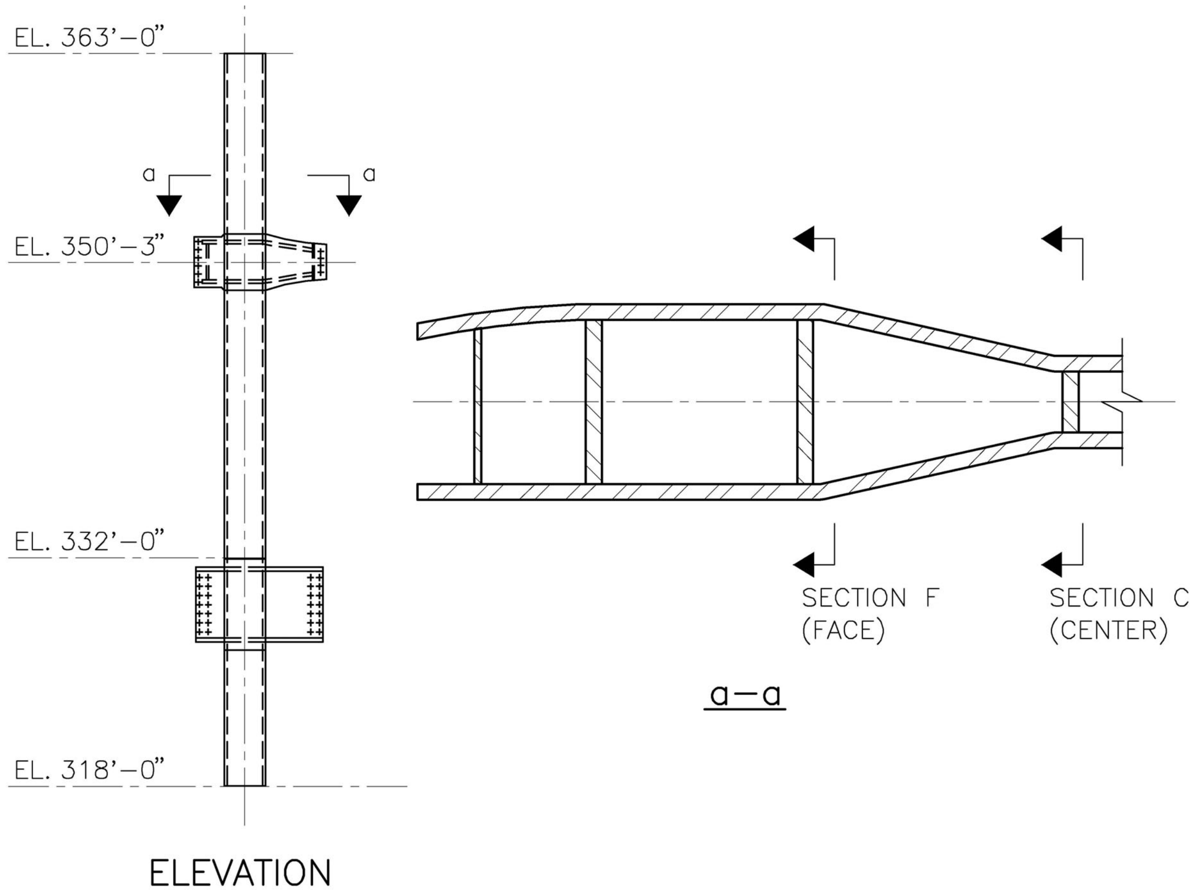
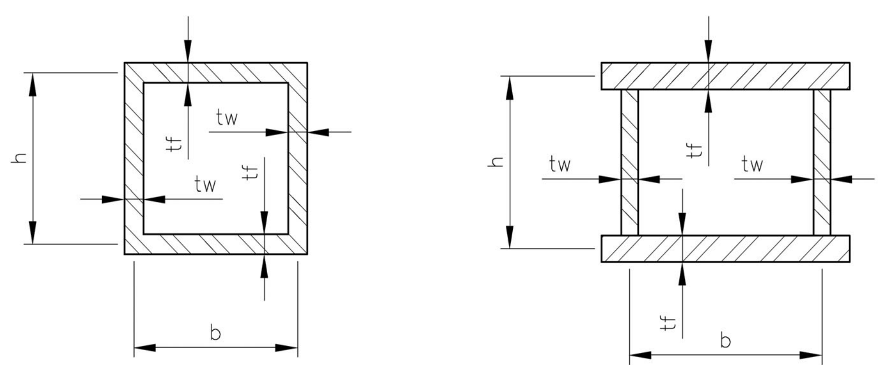


Figure B–12. Column type 6000 with tapered spandrel (taken from drawing book 1, pages 1–A2–27 and 28).

The approximate equation used to calculate J values by the LERA in-house program for a built-up column or box section as shown in Fig. B–13 is as follows:

$$J = \frac{2(bh)^2}{\dfrac{b}{t_f} + \dfrac{h}{t_w}}$$



(a) (b)

Figure B–13. Box section and a built-up column.

In order to minimize the complexity of the model, where the member cross-section was of the type illustrated in Fig. B–13 (a), box column members were defined in SAP *Tube Section*. The remaining built-up box columns (similar to Fig. B–13 b) were defined in SAP *Section Designer*.

For members whose properties are not given in the GFI database, hand calculations or calculations by LERA in-house program were carried out to verify the results from SAP2000 *Section Designer* for at least one section for each member type.

In summary, it was found that SAP2000 *Section Designer* provided section properties in close agreement to LERA calculated properties. In most cases these properties also closely matched with the properties listed in the GFI database. In the cases where SAP2000 results disagreed with the GFI database, the results were reviewed and it was concluded that the SAP2000 calculation provided the correct properties. Therefore the section property results calculated using SAP2000 were used in the WTC-DB and the development of the finite element models of the towers.

Rolled Shape Database

While the majority of the primary members of the WTC towers' super-structure were built-up members, rolled shapes were also used. The rolled shapes specified in the drawings in a number of cases are no longer produced and therefore, are not included in the rolled shape database embedded within SAP2000. Therefore, a rolled shape database was developed using the old nomenclature and section properties. The result was a file named 'Shape Property Table.xls' and it contains three worksheets, 'Database', 'Excel Format', and 'WF Shape Properties from SAP'. The following is a discussion of their contents.

Data contained in 'Database' and 'Excel Format'—Drawing Books 3, 4, and 5 include reference to specific rolled shapes. The referenced shape names were extracted from the above books and assembled into a single reference database for rolled shapes. Most of the section properties were obtained from the Manual of Steel Construction, American Institute of Steel Construction (AISC), Sixth Edition, 1963 (AISC 6th Edition) with few exceptions where cross sections were not included in this edition. Examples of these exceptions include the following:

- Section properties of 14WF455 to 14WF730 were obtained from the Manual of Steel Construction–Load and Resistance Factor Design, American Institute of Steel Construction, Third Edition, 2001 (AISC-LRFD 3rd Edition).

- Section properties of 6CH12, 6CH15.1, 12CH40, 12CH45, and 12CH50 were obtained from the MC-shapes table in the AISC-LRFD 3rd Edition.

- Section properties of 18WF69 were obtained from the Iron and Steel Beams 1873 to 1952, the American Institute of Steel Construction, 1968. 16WF342 was assumed to have the same section properties of 16H342 tabulated in Iron and Steel Beams 1873 to 1952, the American Institute of Steel Construction, 1968.

- For 7x5 tube, Z_x, Z_y, and J were obtained from the AISC- Allowable Stress Design (ASD), 1989, 9th Edition.

- For 2L 3 1/2 × 3 × 1/2in. long leg back to back, the combined properties were taken from SAP's embedded rolled shape database.

Data contained in 'WF Shape Properties from SAP'—For the rolled wide flange shapes, an additional database was created in SAP2000 based on the tabulated shape dimensions from the AISC Manuals as discussed above. Computers and Structures, Inc. provided an MS Excel file named 'Proper.xls' with a macro that allowed the accurate calculation of the section properties for use within SAP2000. This information was then used by SAP2000 *Section Designer* to calculate section properties for built-up members comprised of wide flange sections and added plates.

For calculation of the properties with 'Proper.xls', dimensions of the webs and flanges, as well as the size of the fillet, were input into the spreadsheet. The macro then calculated the section properties based on the input information. The results were shown to be in good agreement with the original tabulated properties.

B.3 DEVELOPMENT OF REFERENCE STRUCTURAL ANALYSIS MODELS FOR THE TOWERS

This section outlines the development of the reference structural analysis models for each of the two towers. Included in this section are descriptions of the structural models, modeling techniques, parametric studies utilized in the development of the models, and a description of the methodology used in exporting data from the relational database presented in Section B.2 into the global models.

The main types of the models developed are as follows:

- Two global models of the major structural components and systems for the towers, one each for WTC 1 and WTC 2; and
- One model each of the typical truss-framed floor and typical beam-framed floor (mechanical level) within the impact and fire regions.

The models are all linear elastic, three dimensional structural analysis models developed using Computers and Structure, Inc.'s SAP2000 Software, Version 8. The models will be used to establish the baseline performance of each of the two towers under gravity and wind loads in the third phase of this project. In addition, these models serve as a reference for significantly more detailed models to be developed independently in other parts of the NIST investigation for the aircraft impact analysis (Project 2) and thermal-structural response and collapse initiation analyses (Project 6).

B.3.1 Global Models of the Towers

Three-dimensional structural analysis computer models of the 110-story above grade structure and 6-story below grade structure for each of the two towers were developed. The global models for the towers consist of the major structural components and systems required to establish the baseline performance of the towers under gravity and wind loads.

In establishing the modeling techniques for the global models, parametric studies were performed to evaluate the behavior of typical portions of the structure (Section B.3.4). In addition, once the models were completed, order-of-magnitude checks were performed for gravity load, wind load, and eigenvalue results to check the accuracy of the models. More refined checks will be done in the third phase of this project on baseline performance analysis.

Components and Systems in the Towers Global Models

The models included all primary structural elements in the towers including exterior columns, interior (core) columns, exterior wall bracing in the basement floors, core bracing at the mechanical floors, core bracing at the main lobby atrium levels, spandrel beams, hat trusses, and rigid and flexible diaphragms representing the floor systems as developed in Section B.3.4 of this report.

Coordinate System, Nomenclature, and Models Assembly Overview

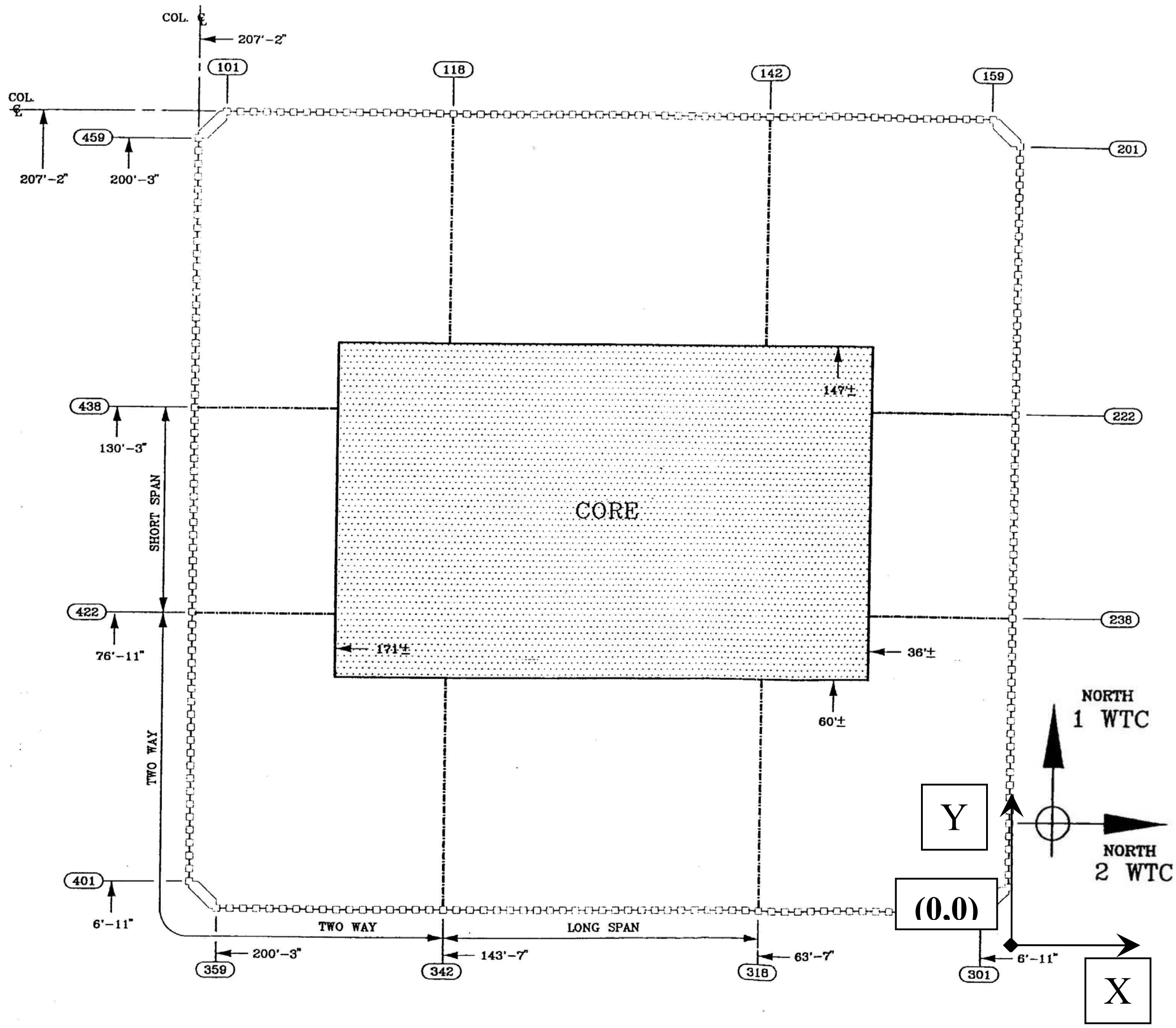
The extent of the data required to assemble the tower models dictated that the relational database capability of the WTC-DB be used (see Section B.2.3). The methodology for the development of the models using the relational database is described in this section.

Coordinate System—The coordinate system for the model geometry was based on the column layout from the original drawings. Figure B–14 shows the location of the X and Y axes for the global models and the floor models. The Z coordinates were based on actual elevations of the towers. The original column numbers were used throughout the models for member identification.

Nomenclature—A standard nomenclature for joints, frame names, and section names for use in the models was established. The nomenclature enables the user to know quickly where in the building a section is located by viewing any given piece of the model. Joint names generally included the column number, tower letter, and floor level. Frame element names generally included the joint name at the 'j' end (second node). Section names were based on the section as described in the drawing book and were repeated for each steel yield strength assigned for that section. Alternatively, where the section was unique to a particular member in the building, sections were named based on the frame member.

As an example, most nodes (or joints) in the tower models were named according to the following format:

- Column number
- Tower letter (A for WTC 1 and B for WTC 2)
- Floor level
- S for column splice nodes only
- J for spandrel splice nodes only



Original drawing used with permission from PANYNJ.

Figure B–14. Global model coordinate axis location.

Figure B–15 illustrates the detailed frame and joint nomenclature for a typical exterior wall panel.

Model Assembly Overview—An overview of the assembly of the data into the tower model is described herein along with an expanded section on the programmatic assembly of the models.

Following a basic study of modeling techniques and testing of SAP2000, Version 8 input format and capabilities, it was determined that the best approach was to divide the model into six main parts and thenassemble them into a unified model. Manipulation of these individual parts was more efficient than attempting to build the whole model simultaneously.

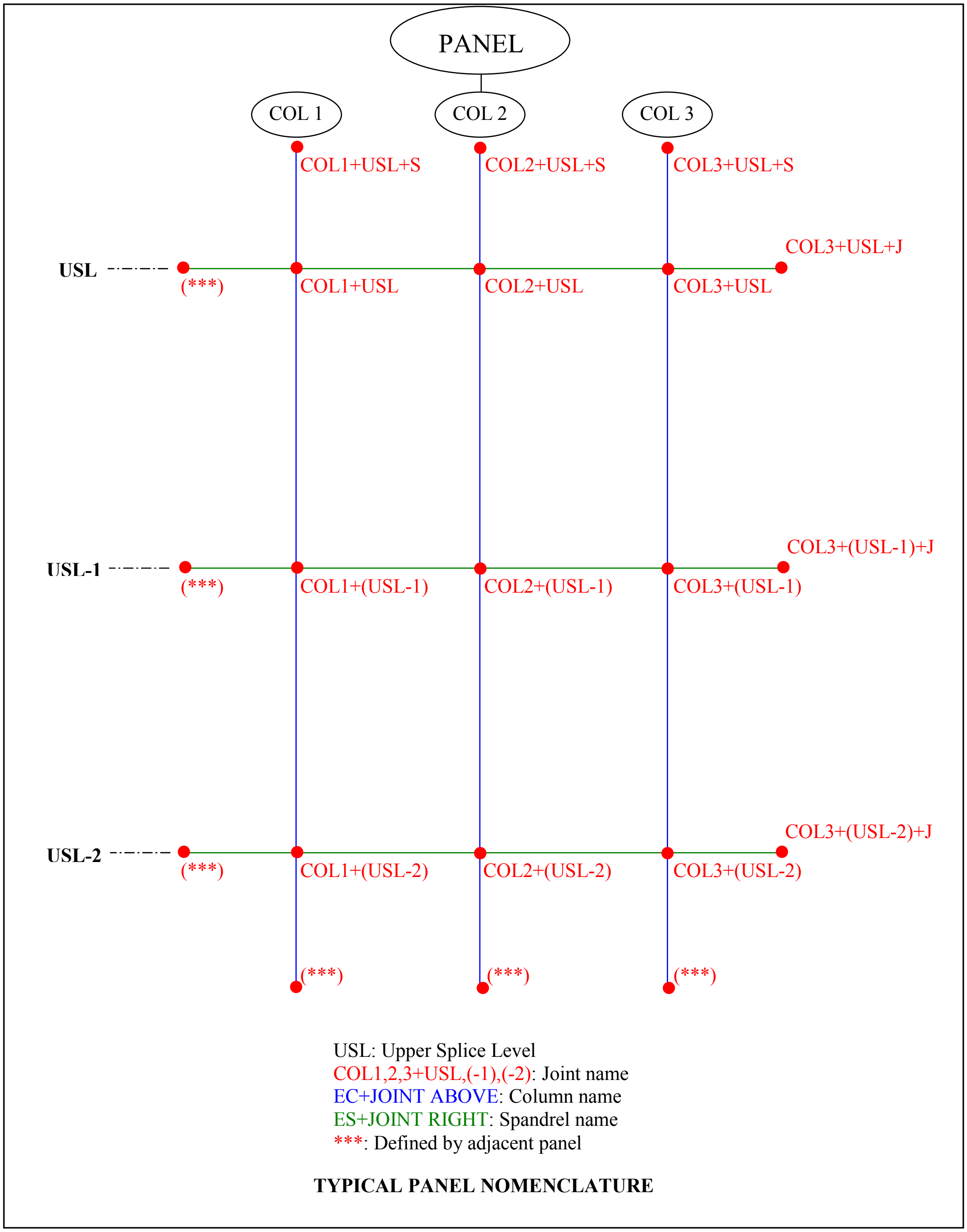


Figure B–15. Typical exterior panel nomenclature.

The six initial models were:

- Core columns
- Exterior wall, foundation to floor 7
- Exterior wall trees (floors 7 to 9)
- Exterior wall, floors 9 to 106
- Exterior wall, floors 107 to 110
- Hat truss

For the core columns and exterior wall at floors 9 to 106; most of the analysis input files were generated from queries of the WTC-DB. The other four parts of the model were assembled primarily in a more conventional manner.

Core columns and exterior wall panels (floors 9 to 106) were the greatest data-intensive challenges in the model development. Both areas included a large number of frame members and section and material property variations. The query files were used to gather the necessary data, and then simple computer programming was used to convert the data into the SAP input file format. Four main input tables for the SAP input file were developed programmatically:

- Joint coordinates table
- Connectivity-frame/cable table
- Frame section properties tables
  - Frame section properties 1—general
  - Frame section properties 5—nonprismatic
  - Section designer properties 04—shape I/wide flange
  - Section designer properties 05—shape channel
  - Section designer properties 11—shape plate
- Frame assignments table

The remaining data is added directly in the SAP model:

- Material properties
- Frame local axis
- Joint restraint

- Insertion point
- Constraint
- Gravity and wind load assignments

After the joint coordinates, connectivity, frame section properties, and frame assignments were complete for the six parts, the individual models were combined into a unified model. Rigid diaphragms, flexible diaphragms, core bracings, gravity loads, wind loads, and masses were then added to the unified model. After assembly of the model, the assignment of properties for selected model elements was spot-checked and the model was executed to verify its performance.

The development of the WTC 1 and WTC 2 models has been separate and consecutive endeavors. The lessons learned in the assembly of the WTC 1 model were applied to the development of the WTC 2 model. While there were only minor differences in the basic structural systems of the two towers, there were significant differences in section properties, material properties, and additional column transfers at the lower levels in WTC 2.

Isometric views of the complete WTC 1 model are illustrated in Fig. B–16. Elevations of the complete WTC 2 model are illustrated in Fig. B–17. A summary of the size of the global models of WTC 1 and WTC 2 is presented in Table B–2. The following presents the details of each of the six parts used in the development of the unified global models for WTC 1 and WTC 2.

Core Columns Modeling

Core column coordinates were tabulated based on the structural drawings. Column locations were typically referenced at their centerlines. Columns on lines 500 and 1000, however, were located in plan drawings along most of their height according to the face of the column to which the floor trusses frame (i.e., WTC 1 north face for 500 columns and south face for 1000 columns). The centerline of these columns was based on their dimensions given in the drawing books. Where these column centerlines varied along the height of the towers (typically 1 1/2 in. between three-story pieces), a representative location was chosen to define the column node. Thus, the column coordinate at floor 106 was used as a constant along the tower height because at this level, these columns align with the hat truss above.

The spandrel centerline elevation was selected as the representative floor elevation for exterior columns and used also for core columns. If there were no spandrels in exterior panels, reference elevations were used for the core columns.

There were over 5000 nodes in the core column model. This amount of data required that the *Interactive Database* input table be set up using a macro. These data were converted to text file format and later imported into SAP. Built-up sections were defined as *Section Designer* sections, and wide flange shapes were defined directly from "SectionWF1.pro" file (see Section B.2.5). All section names were identical to those in the database. Around 1280 *Section Designer* sections were defined in this model and imported through *Interactive Database* function of SAP2000 to the model.

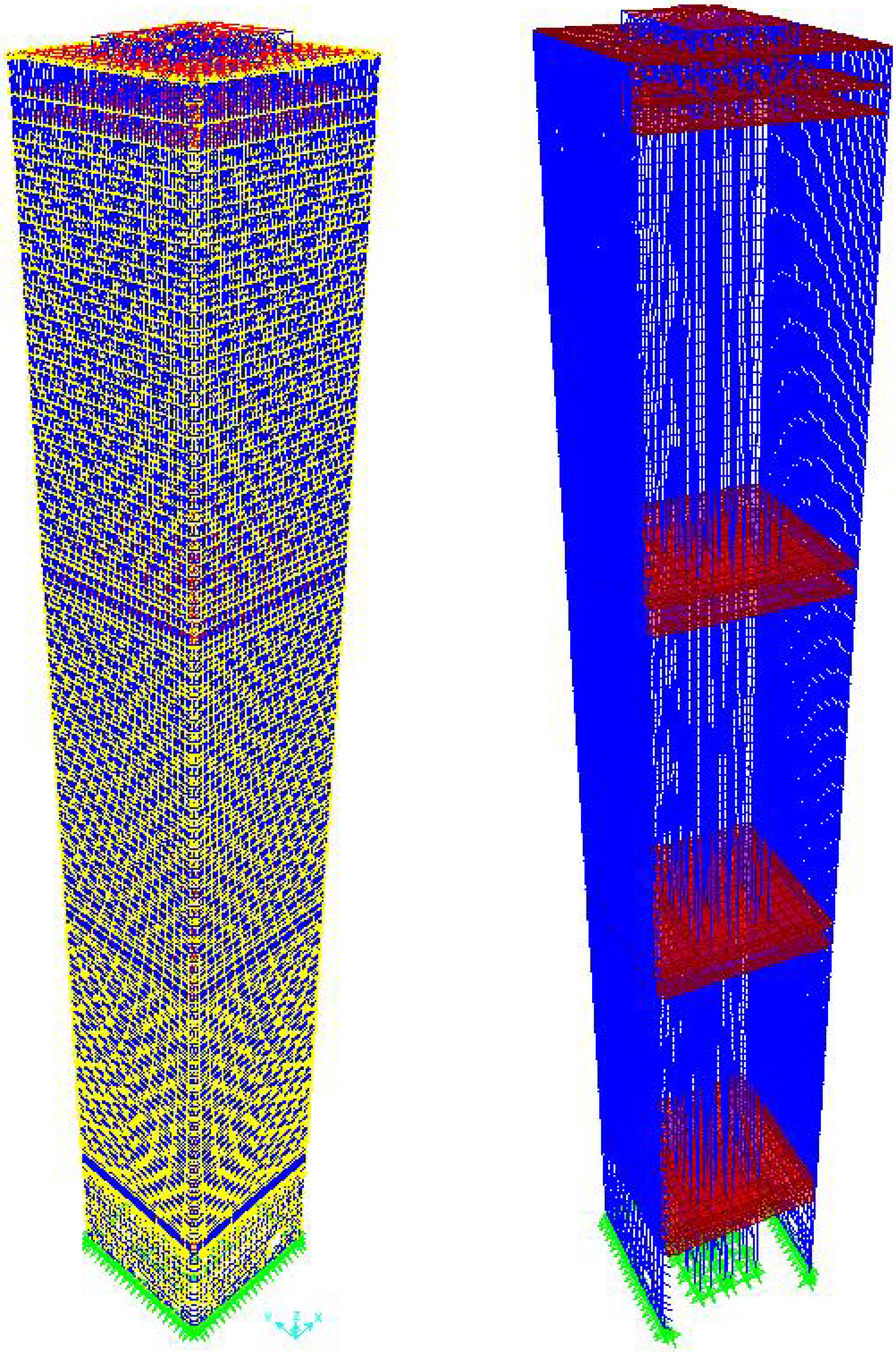


Figure B–16. Rendered isometric views of the WTC 1 model.

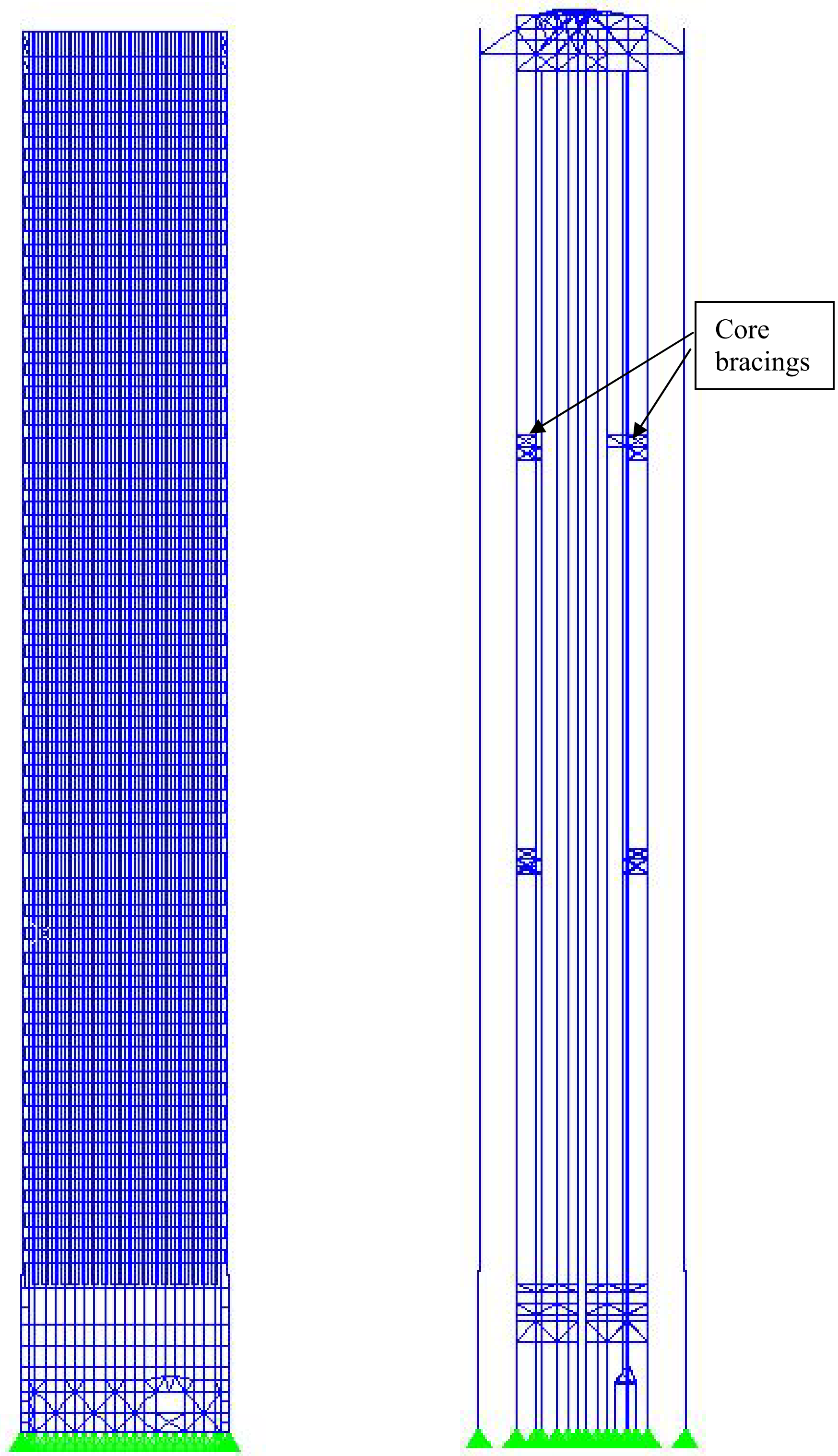


Figure B–17. Frame view of the WTC 2 model: exterior wall elevation and interior section illustrating the core columns, core bracing, and hat truss.

Table B–2. Approximate size of the reference structural models (rounded).

Model	Number of Joints	Degrees of Freedom	Number of Frame Elements	Number of Shell Elements	Total Number of Elements
WTC 1 global model[a]	53,700	218,700	73,900	10,000	83,900
WTC 2 global model[a]	51,200	200,000	73,700	4,800	78,500
Typical truss-framed model	28,100	166,000	27,700	14,800	42,500
Typical beam-framed model	6,500	35,700	7,500	4,600	12,100

a. Model does not include floors except for flexible diaphragms at 17 floors as explained later.

The core columns were defined as frame members spanning from node to node at the representative floor elevations. Splices in core columns occurred typically 3 ft above the floor level. In the models, however, the splice was considered to occur at the floor level, and nodes were only defined at these levels (i.e., typically at spandrel centerlines). Most three-story column pieces are unique, as tabulated in WTC-DB (Drawing Book 3). A section for each three-story piece was defined and then assigned to each of the three frame members that make up that column. Using the SAP shading feature to graphically show the section on the model, each frame was rotated to its proper orientation based on the structural drawings.

In the as-designed drawings, there were strap anchors connecting the core columns to the concrete floor slab to provide lateral bracing for the column. At floor 26 of WTC 1 the straps at column 901 were removed during a renovation project that was engineered by a firm other than LERA (see Section B.2.4). The loss of the straps at this location has been included in the model by releasing the column from the diaphragm in the direction of the straps.

Exterior Wall, Foundation to Floor 7 Modeling

The models of the exterior wall up to elevation 363 ft were developed manually, assigning joints and members connectivity as shown in the drawings. The elevation drawings show that below elevation 363 ft, columns were typically spaced at 10 ft and braced with spandrels and diagonals. Joints were defined at all locations where diagonals braced the columns. However, when coordinates were not given in the drawings, joint coordinates were determined based on the geometry of the diagonal. Details in WTC Drawing Book 1 show that the column-diagonal intersections had continuity. Joints at elevation 253 ft (level B-5) were defined only where the diagonals connect to the columns, since the tower floor did not frame into the exterior spandrels at that floor.

Where noted in elevation drawings, spandrel centerline elevations were used to define joint coordinates. Additionally, joints were defined at the spandrel splice midway between two columns at elevation 350 ft 3 in. (floor 3) and at elevation 329 ft 3 in. (floor 2) to allow for section type transitions.

The majority of the elements at these levels were defined as *Section Designer* sections, except for box shapes which were defined as “Box/Tube”. Channel shapes were defined directly from “SectionWF1.pro” file (see Section B.2.5). All section names were identical to those in the database. Around 200 sections were defined in this model using the *Interactive Database* function of SAP2000, which was used to import data into SAP2000.

Typical columns were connected from bottom to top and typical spandrels were connected from left to right. Frame names followed the nomenclature description presented earlier. The SAP2000 program allows assignment of rigid zone factors to frame end offsets to account for the overlap of cross sections. At the intersection of columns and spandrels, 100 percent rigidity for the column and the spandrels were assigned due to the large size of both columns and spandrels. Using the SAP shading feature to graphically show the section on the model, each frame was rotated to its proper orientation based on the structural drawings.

Refer to Fig. B–18 for a frame view and rendered view of the exterior wall (foundation to floor 9) of the WTC 1 model. The figure also shows the core columns and core bracings.

Exterior Wall Trees (Floor 7 to 9) Modeling

The panels of the exterior wall between elevation 363 ft and elevation 418 ft 11 1/2 in. are called exterior wall trees. At the exterior wall trees, the typical exterior wall columns transitioned from a spacing of 10 ft to a spacing of 3 ft 4 in. A typical exterior wall tree panel is shown in Fig. B–19. Each panel was divided into five different levels; level B, C, D, E, and F. For each panel in the model, the three exterior columns from above elevation 418 ft 11 1/2 in. continued down to level D. At that level, the three columns were connected by a horizontal rigid element to become one member, which extended down to elevation 363 ft.

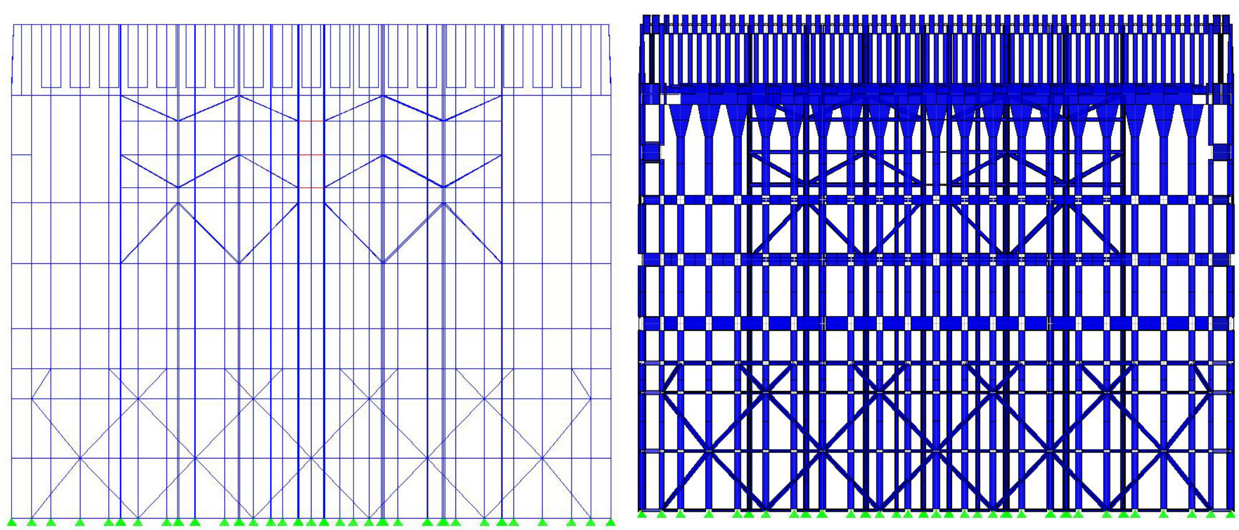
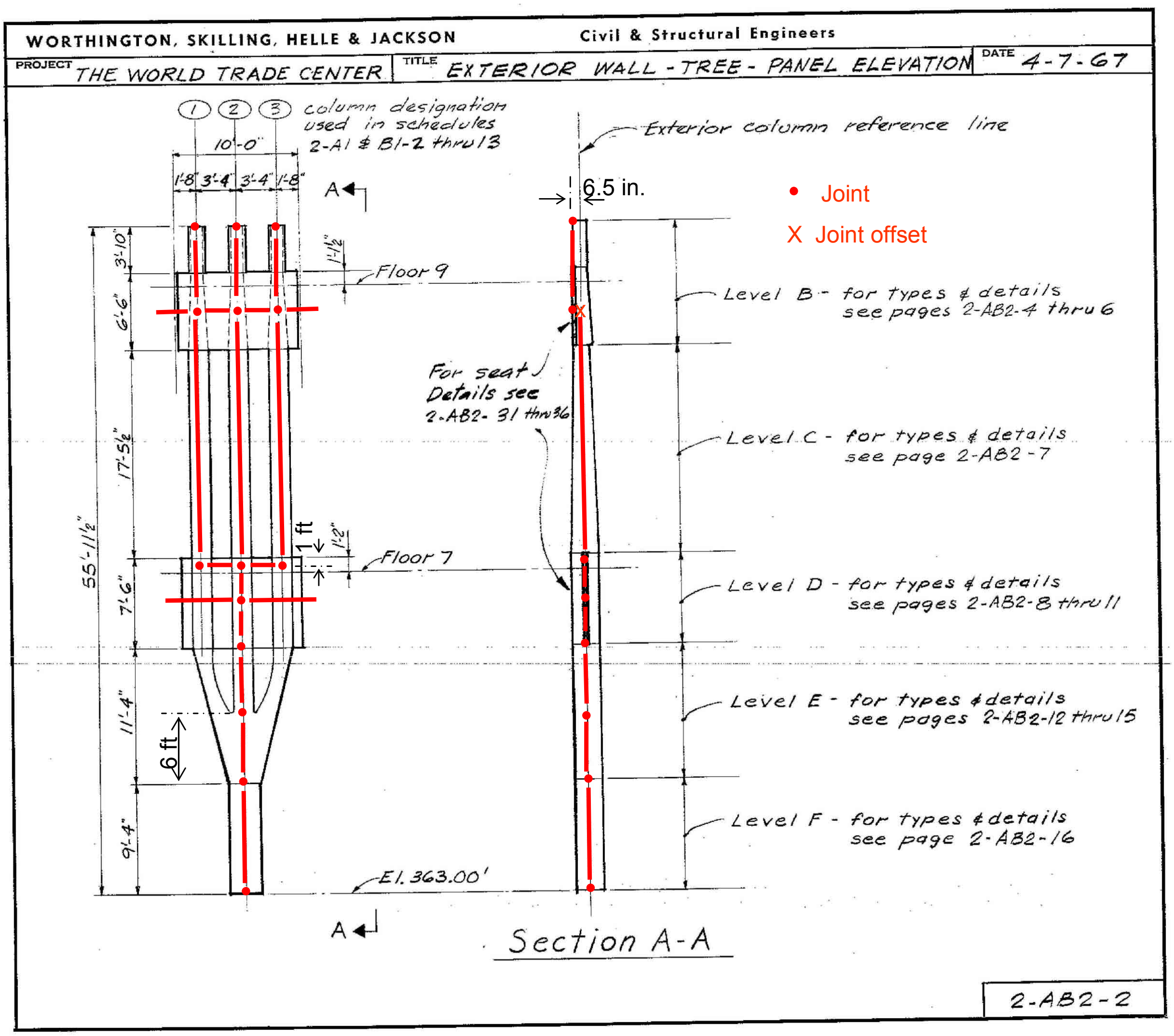


Figure B–18. Frame view and rendered view of the WTC 1 model (foundation to floor 9).

In the model, the tree was also the location where the column insertion point transitioned from the inside face (at the spandrel) of the upper column to the centerline of the lower column. Between levels B and D (see Fig. B–19), the location of the spandrel transitioned from 6 1/2 in. offset from the exterior column reference line to the center of this reference line. Within the floor 9 spandrel, the exterior columns taper; however, in the model, the tapering of the columns was not included because frame end length offsets were assigned to the columns to account for the rigidity of the spandrels.

Through the height of level C, the box-shaped columns tapered (Fig. B–19). In the model, non-prismatic members were used to model the tapering columns. The columns start to taper at the bottom of the spandrel at level B, and cease to taper at the top of the spandrel at level D. The dimensions of the columns at the spandrel edges were defined in the drawing book. In the model, the column extended from the centerline of the spandrel at level B to 1 ft below the top of the spandrel at level D (see discussion for level D below). Therefore in order to obtain the correct section properties along the length, the dimensions of the section at the joints were interpolated based on the dimensions of the section at the spandrel edges shown in the drawing book. The section properties of the tapering column were assumed to vary linearly between the two sections. Frame end length offsets were assigned to the columns to account for the rigidity of the spandrel at level B and the one foot dimension at level D.



Original drawing used with permission from PANYNJ.

Figure B–19. Exterior wall tree panel (taken from Drawing Book 2, page 2-AB2-2).

At level D, two transitions occurred in the model. The first transition was for the exterior columns, where the three columns coming down from level C were connected by a horizontal rigid element to become one member at the bottom of the tree. This frame member consisted of the three exterior columns and the spandrel plate. Another horizontal member of the same section properties with the spandrel plate was also defined and connected between the neighboring exterior wall trees. This member connected the neighboring exterior wall trees and provided lateral bracing for the columns. Frame end length offsets were assigned to the spandrel to account for the overlap of the spandrel plate with the frame member, which also included the spandrel plate. The transition of the three members into one member was assumed to occur at one foot below the top of the spandrel at level D to account for the fact that the spandrel becomes engaged with the exterior columns after being connected to the exterior columns for a certain distance. Hence, the joints were defined at one foot below the top of the spandrel at level D.

There was a second transition at level D (Fig. B–19). The nodes for the exterior wall columns were typically defined at 6 1/2 in. offset from the exterior column reference line. But for the joints at and below level D in the exterior wall tree, the joint coordinates were defined along the exterior column reference line. As a result, for the column member that framed between the nodes at levels B and D, a joint offset of 6 1/2 in. was assigned at the top of the member, while no offset was assigned at the bottom. The column therefore remained a vertically straight element while being connected to nodes that were not aligned vertically.

At level E, the exterior columns tapered and had two different types of cross section (Fig. B–20). For each panel, the exterior column transitioned from Section b–b in Fig. B–20 into a box-shaped column (Section c–c in Fig. B–20). The location of the transition between the different types of cross section varied for different column types from 5 ft 8 in. to 6 ft 4 in. measured from the bottom of level E. In the model, the transition was assumed to be at 6 ft measured from the bottom of level E. For each panel, the exterior column at level E was modeled as two nonprismatic members. The top section of the first non-prismatic member consisted of three box-shaped columns and a middle plate, while the bottom section was a box-shaped column (Section c–c in Fig. B–20). The properties were assumed to vary linearly between the two sections. The second nonprismatic member was a tapering box shaped column (Section c-c in Fig. B–20), and again, the properties were assumed to vary linearly between the two sections. At level F, the exterior wall tree columns were prismatic box-shaped columns.

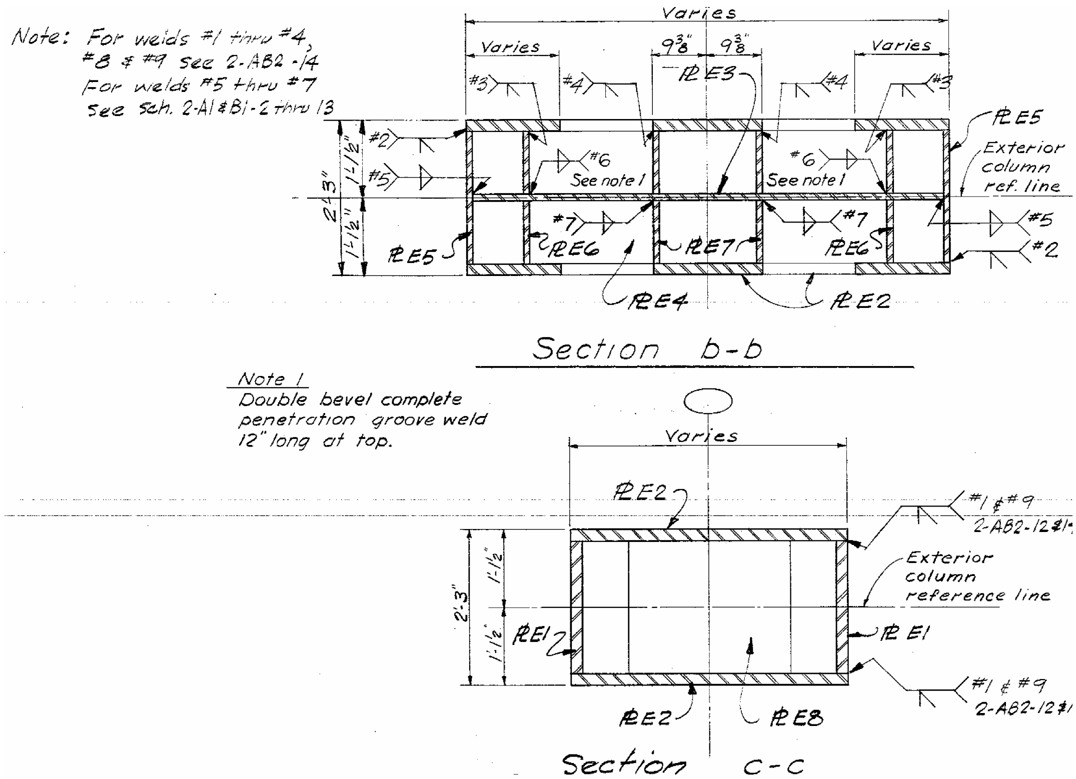
The final model of a typical tree is illustrated in Fig. B–21.

Exterior Wall (Floor 9 to 106) Modeling

In plan, column and spandrel members connected at nodes located at the outside face of the spandrel, 6 1/2 in. from the exterior column reference line (see Fig. B–22). The columns were offset horizontally, or 'inserted' at this node, using an insertion point located at the centerline of plate T3. Insertion points were not adjusted for spandrel thickness. With this modeling, gravity and wind loads can be applied at the spandrel location.

In elevation, the columns and spandrel members connected at the spandrel centerline, typically 12 1/2 in. below the reference floor elevation (Fig. B–22). The spandrels were then located correctly without the need for offsets to be defined. The effect of applying loads at both the spandrel centerlines and the reference floor elevations was studied, and it was found that it has a negligible difference in spandrel stresses.

For typical exterior wall panels (i.e., three columns wide by three stories high), nodes at five elevations were defined. The models included nodes at the three representative floor levels (defined at the spandrel centerlines) as well as the upper and lower column splices. Diaphragms were assigned to all nodes at floor levels where concrete slabs exist, to represent the high in-plane stiffness of the concrete floor slabs.

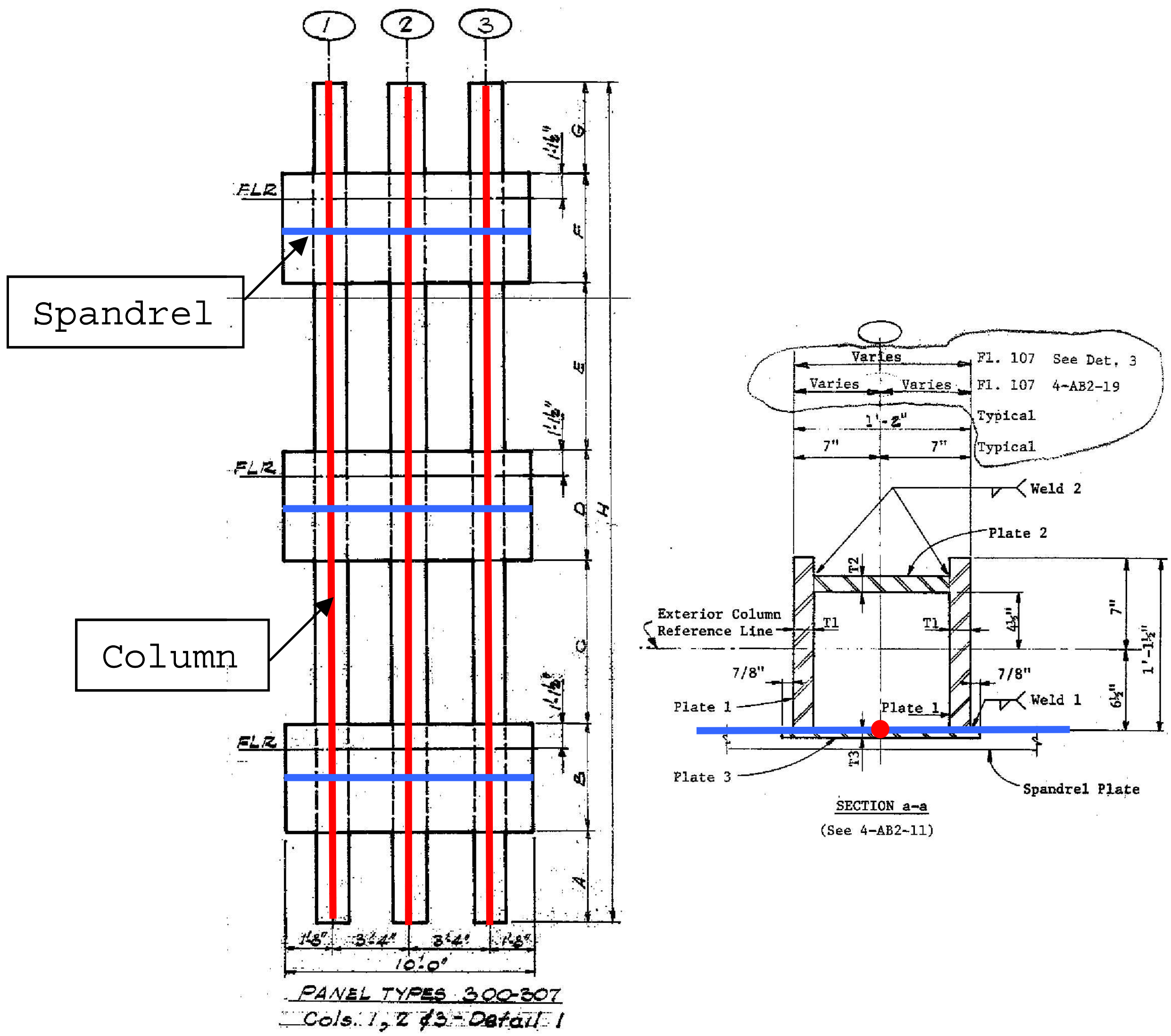


Original drawing used with permission from PANYNJ.

Figure B–20. Exterior wall tree: as-built cross sections for level E (taken from Drawing Book 2, page 2-AB2-13).

Original drawing used with permission from PANYNJ.

Figure B–21. Frame view and rendered view of an exterior wall tree.



Original drawing used with permission from PANYNJ.

Figure B–22. Typical WTC tower exterior wall panel.

The SAP2000 program allows assignment of rigid zone factors to frame end offsets to account for the overlap of cross-sections. In the global model, 50 percent rigidity for the column and 100 percent rigidity for the spandrels were assigned for the typical exterior wall panels to match the lateral deflection of the detailed shell model of the panel based on the parametric study results (see Section B.3.4). It was also found that, due to the relatively large depth of the spandrels and the close spacing between the columns, the spandrels contribute to the axial stiffness of the columns in the panels. This contribution was estimated to range from 20 percent to 28 percent increase in the vertical stiffness of the panels. Therefore, a frame property multiplier for the exterior wall column's cross-sectional area was used to provide a 25 percent increase in columns' axial stiffness (see Section B.3.4).

For exterior wall corner panels, 25 percent rigidity for the column and 50 percent rigidity for the spandrels were assigned based on the parametric study results (see Section B.3.4). Also, an area modifier was used to provide a 25 percent increase in the axial stiffness of the two continuous columns of the corner panels (Section B.3.4). No modifier was used for the 100, 200, 300, and 400 series intermittent columns.

Exterior column types were defined in Drawing Book 4. A few types (100 series typical, 300 series at mechanical floors, and 400 to 500 series at corners) repeated extensively throughout the building, with steel yield strengths that vary from 36 ksi to 100 ksi. Since SAP does not allow for the assignment of material properties at the member assignment stage, the number of different steel strengths was determined for each exterior column type, and sections were defined for each. The section name included the section number and the yield strength as tabulated in the drawing books.

Typical spandrels and corner panels were defined as rectangular shape and *Section Designer* section with stiffener, respectively. The top and bottom stiffener of each corner spandrel were included in both the parametric study and the global models. The detail shows that the stiffeners were 6 in. plates of thickness matching plate T2 in the corner column.

Exterior Wall (Floor 107 to 110) Modeling

Spandrel depths varied at floors 108 and 110. A weighted average of spandrel depth was determined in order to define the average centerline elevation of the spandrels and, therefore, the node elevation for the entire floor.

For the 7 × 5 structural tube sections that were used in these floors, sections from the current AISC Manual were assigned, and modification factors of 1.04 were applied to the section properties. The modifiers were used to match the section properties from the 6th Edition AISC Manual.

The exterior wall members from floors 107 to 110 were typically rolled shapes with F_y =42 ksi or F_y =50 ksi. Where not shown in the drawings as F_y =50 ksi, F_y =42 ksi was used.

Hat Truss Modeling

In both WTC 1 and WTC 2, a truss system referred to as a ‘hat truss’ was constructed between floor 107 and the roof. The hat truss system was intended to support the load of the antenna on top of the tower and to interconnect the exterior walls to the core. The hat truss was made up of eight trusses spanning perpendicular to the long-direction of the core and six trusses spanning perpendicular to the short-direction of the core (refer to Figs. B–23 and B–24).

Frame members between floors 107 and 110 were assigned to the model according to plan and elevation drawings of the hat truss. Node locations were set to coincide with the centerline of spandrels at the exterior wall. Columns, diagonals, and beams were included in the model. All columns and diagonals shown in drawings SA/B–400 through SA/B-404 were included in the model. Floor beams that did not participate in the hat truss system were not included in the model, unless they were used to transfer truss chords or core columns. Flexible floor diaphragms were used in this area.

Coordinates were generally not given at floor 109, as this level does not contain a complete concrete floor slab. The geometry of the diagonals, columns, and beams was used to determine the location of the node where the diagonal would intersect floor 109. Unless otherwise noted in the drawings, diagonals and

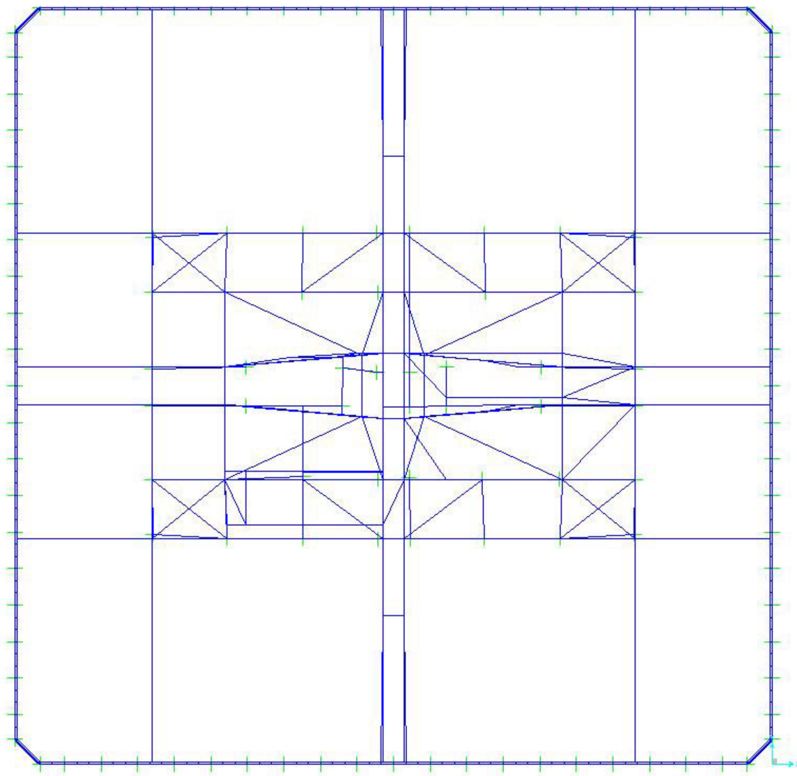


Figure B–23. As-modeled plan of the WTC 1 hat truss.

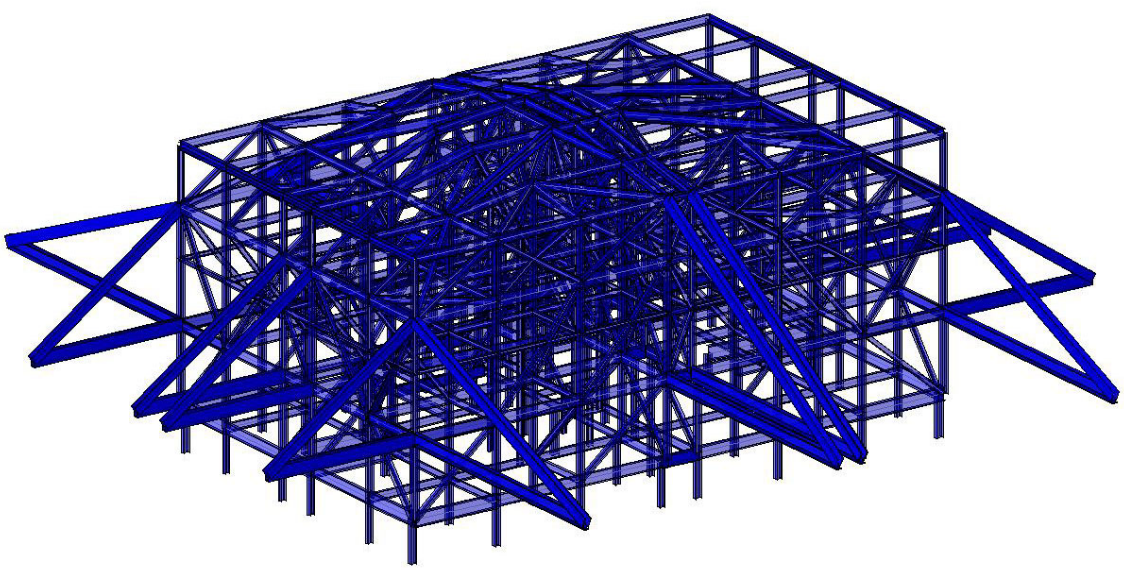


Figure B–24. Rendered 3–D model of the WTC 1 hat truss (prior to assembly in the unified model).

columns were assumed to be non-composite and floor beams were assumed to be composite. Hat truss diagonals, main chords, and main columns were modeled with continuous joints. Hat truss beams, however, had pinned ends.

Flexible and Rigid Floor Diaphragm Modeling

For most floors, rigid diaphragms provide for a sufficiently accurate representation of the flow of forces and deformations for global structural response. This is a customary engineering practice. In cases where the flow of forces and deformations would be affected significantly by the use of rigid diaphragms, the floors were modeled as flexible diaphragms.

The floor models described in Sections B.3.2 and B.3.3 were used to develop the flexible diaphragm stiffness utilized within the global models. Section B.3.4 outlines the study for the determination of the in-plane diaphragm stiffness of the detailed floor models, using that in-plane stiffness to arrive at an equivalent shell element floor model. The equivalent shell element floor was used to represent the in-plane floor stiffness in the global model. The shell elements attached to all exterior wall columns and core columns.

Flexible diaphragms were used at the floors of the towers in the core of the atrium area, in the mechanical floors, and in the floors of the hat trusses. The floors modeled using flexible diaphragms are floors 3, 4, 5, 6, 7, 9 (atrium levels); 41, 42, 43, 75, 76, 77 (mechanical levels); 107, 108, 109, 110, and roof (hat truss region) of both towers.

Initial Verification of Global Models

Several steps were taken to verify the model input. SAP2000 Version 8 offers a ‘shading’ option once a model has been built with frame section assignments. This allows the user to view the members as the program has interpreted their input. The shading option was helpful for using section-designed shapes, and for verifying the orientation (i.e., local axes) of members. Note that shading is not correct when two *Section Designer* sections are used in non-prismatic members, so orientations for these sections were verified by reviewing their local axis member properties. The work was independently reviewed by engineers not associated with the initial model development.

Once the models were completed, checks for gravity loads, wind loads, and eigenvalue results were performed. The overall performance of the tower models under these loads was found to be reasonable by checking deformations, stresses, reactions, etc. More refined checks will be done as part of the third phase of this project on baseline performance analysis. The natural periods will be calculated using mass properties estimated from realistic loads on the towers as part of the baseline analysis. Calculated natural periods will be compared with the measured periods of WTC 1.

B.3.2 Typical Truss-Framed Floor Model—Floor 96A

In order to select the typical truss-framed floor within the expanded impact and fire zones of both towers, the drawings for floors 80 to 100 were reviewed to identify structural similarities. It was found that floor 96 of WTC 1 (96A) represented the typical truss-framed floor in the expanded region for WTC 1 (floors 89A to 103A). The lone exception in this region of WTC 1 was floor 92 which had an increased dead load capacity required for the support of secondary water lines.

Floor 96A was also representative of the typical truss-framed floor in the expanded region for WTC 2 (floors 74B–88B). Specifically, floor 96A was similar to the truss framing at floor 74B and floors 84B through 88B. Floors 78B and 79B were sky lobby and upper escalator floors, respectively. Both contained long span trusses which were similar to floor 96A, but also contained beam-framed floor construction in the entire short span area (where the escalators were located). Floors 80B through 83B had beam framing in place of a single truss panel in the short span area, while the remaining area contained trusses which were similar to floor 96A.

Based on the above discussion, floor 96 of WTC 1 was selected as the overall representative truss-framed floor for the majority of the expanded impact and fire zone in both towers and is described in the following sections (see Fig. B–25). An isometric view of the typical truss-framed floor model is illustrated in Fig. B–26. Table B–2 includes a summary of the size of the 96A floor model. The following presents the major structural systems and components of the truss-framed floor model.

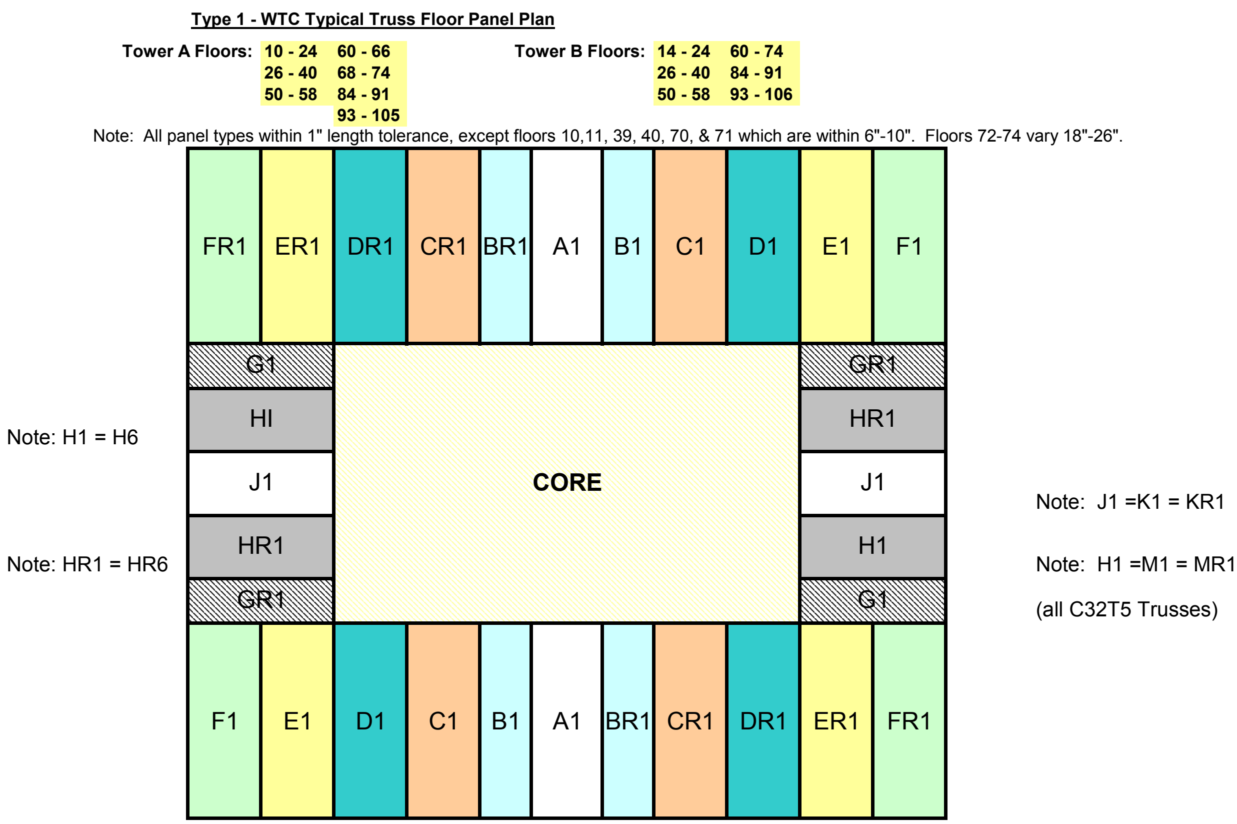


Figure B–25. Typical truss-framed floor panels arrangement.

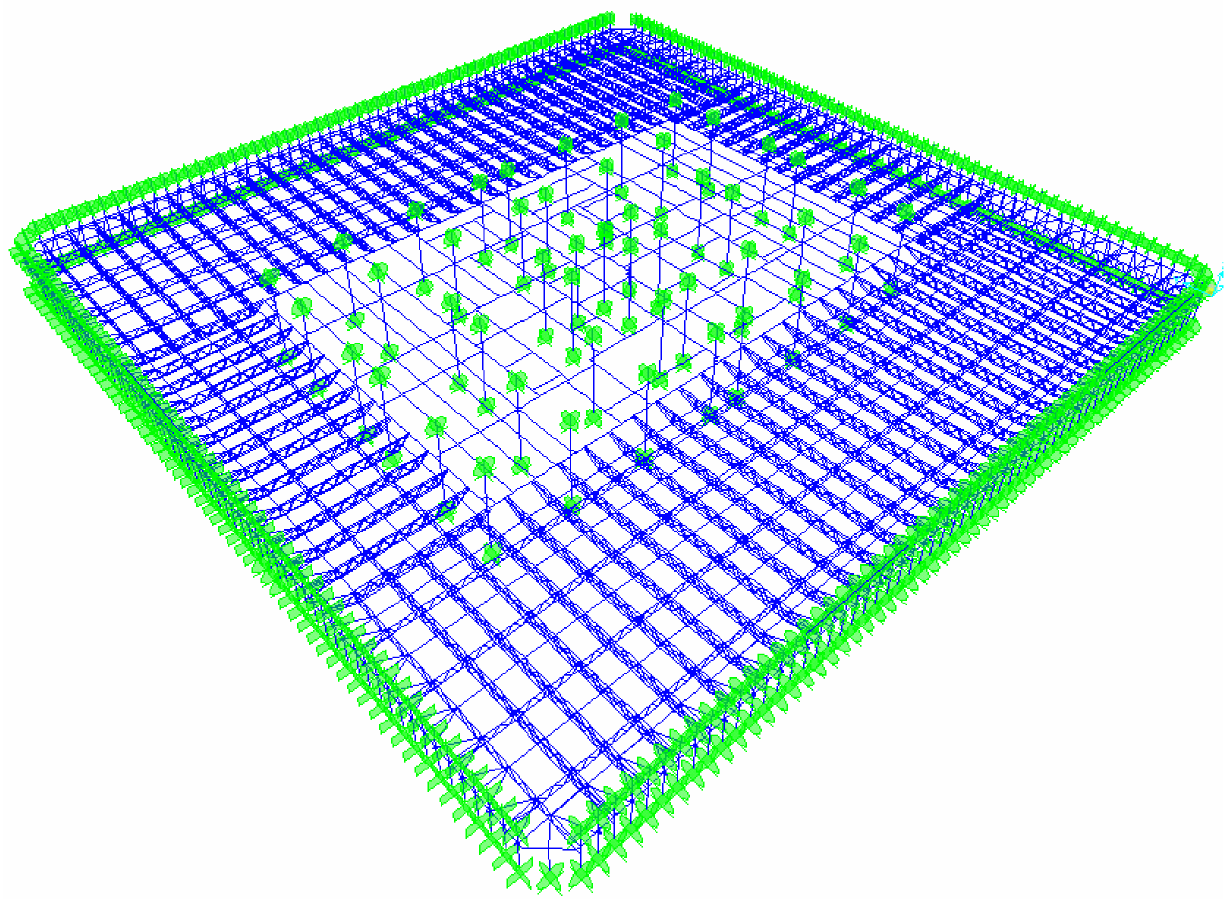


Figure B–26. Typical truss-framed floor model (floor 96A), slab not shown.

Primary Trusses

The primary trusses consisted of double angle top and bottom chords which were 29 in. out-to-out of the chords. The trusses acted compositely with a 4 in. concrete slab on 1 1/2 in. metal deck. For a typical long-span truss, C32T1, the top chord consisted of two angles 2 by 1.5 by 0.25 in., short legs back-to-back (SLB), and the bottom chord consisted of two angles 3 by 2 by 0.37 in., SLB. The distance between the centroid of the two chords was calculated to be 28.05 in. The distance from the centroid of the top chord to the neutral axis of the transformed composite slab with top chord was calculated to be 1.93 in. The sum of (28.05 + 1.93) is 29.98 in. (Fig. B–27). In the model, therefore, 30.0 in. was taken as the typical distance between the top and bottom chords for both short- and long-span primary trusses.

In the long-span truss zone, the two individual primary trusses, which were part of the same floor panel and attached to the same column, were separated (typically) by a distance of 7 1/8 in. At the joint between panels, the distance between the abutting long-span trusses was 7 1/2 in. Therefore in the model, 7 1/2 in. was used as the spacing between all long span primary trusses. In the short-span truss zone, two individual trusses which attached to the same column were separated by a distance that varied between 4 7/8 in., 5 in., and 5 1/4 in. In the model, the typical spacing between all short-span double trusses was 5 in. The long span trusses in the two-way zone had an as-modeled length of 58 ft 10 in. while the long span trusses in the one-way zone had an as-modeled length of 59 ft 8 in.

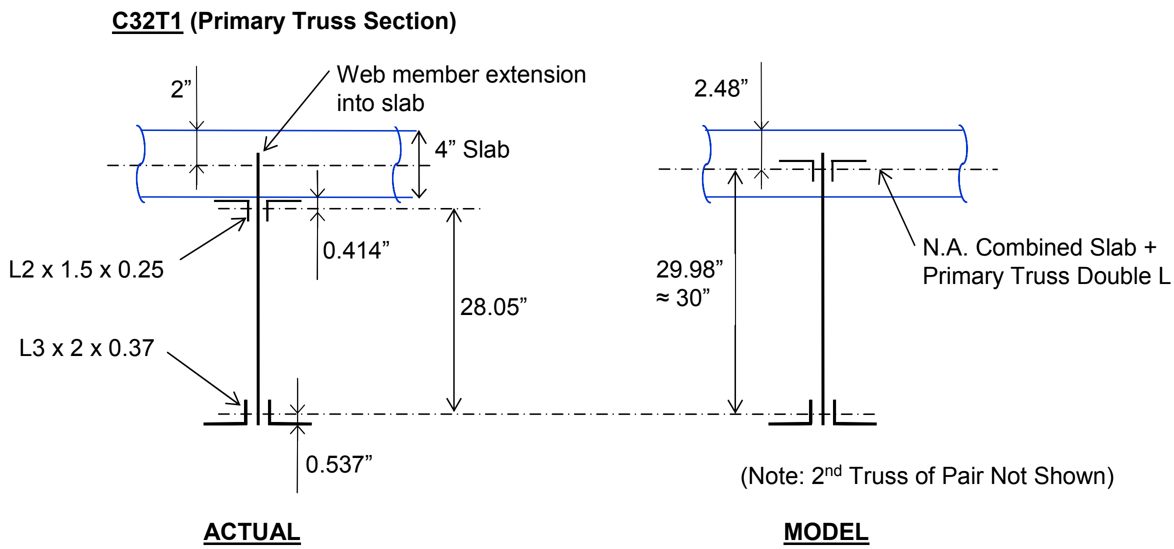


Figure B–27. Typical primary truss cross-section, as-built and as-modeled transformed truss work points.

The diagonal web bars for the primary trusses were most often 1.09 in. diameter bars. Therefore, for double angle shapes in the primary trusses, 1.09 in. is taken as the distance between the two angles. This holds true for primary trusses where bar diameters varied between 0.92 in. and 1.14 in.

The as-built truss diagonals had end fixity, but were considered pinned for the analysis. Pinning the diagonals is conservative and provides an upper bound of the gravity load stresses. To mitigate the effect of the pinned member approach, end length offsets were used for the truss diagonals to compensate for the difference in the as-built diagonal unbraced length and the model unbraced length. The as-built unbraced length for a typical diagonal in a primary truss was 32.4 in., while the modeled member length was 36.05 in., and therefore, an end offset of 1.8 in. was used at both ends. Similarly, for the bridging trusses, the actual unbraced length for a typical diagonal of a bridging truss was 29 in., while the modeled length was 30.66 in. Therefore, an end offset of 0.83 in. was used at both ends. A rigid zone factor of 100 percent is used for all offset zones.

In the model, the deck support angles, typically 3 by 2 by 0.75 in. were located in the same plane as the combined truss top chord and composite slab centroid.

Bridging Trusses

The bridging trusses were 24 in. deep, edge-to-edge, with double angle chords. For a typical bridging truss, 24T11, the top and bottom chords consisted of two angles 1.5 by 1.25 by 0.23 in., SLB. The distance between the centroid of the two chords was 23.26 in. The distance used as the offset between the top and bottom chords for all bridging trusses was taken as 23.25 in. (Fig. B–28). The distance between the work points of the top chord of the bridging truss and the top chord of the primary trusses and equivalent slab plate for 24T11 was calculated to be 3.39 in. This distance was selected for all bridging trusses to be 3.375 in. As in the as-built structure, the bridging truss was not connected along its length to the slab shell elements in the model. At the intersection of the top chords of the primary and the bridging trusses, the intersection was modeled using vertical rigid links, connected in turn to the slab shell elements representing the concrete slab.

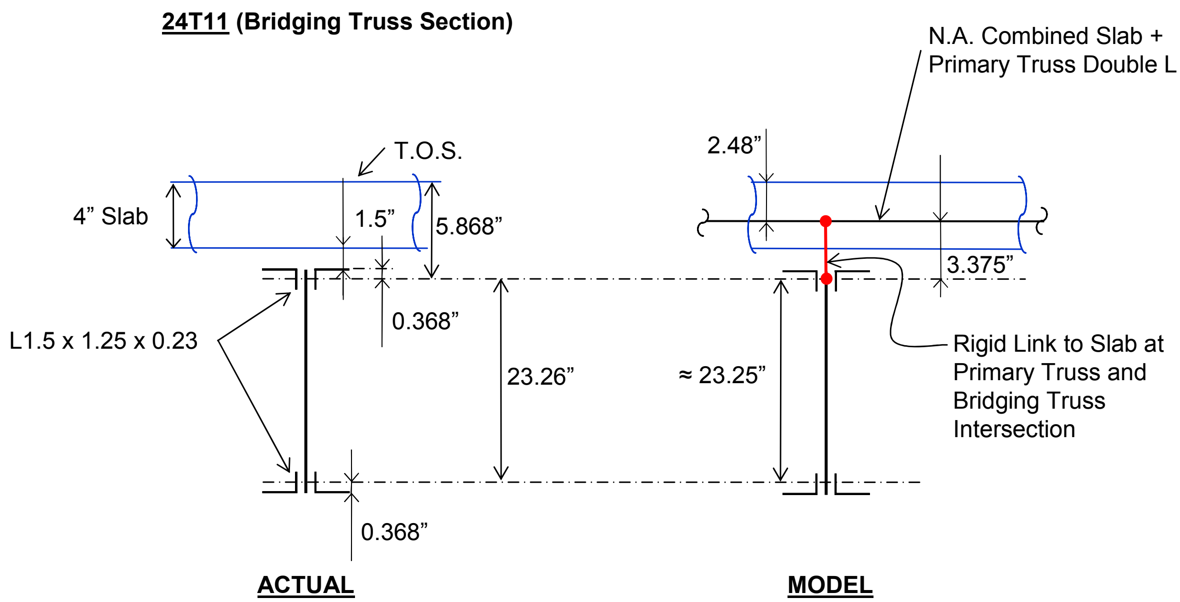


Figure B–28. Typical bridging truss cross–section, as-built and as-modeled transformed truss work points.

The bottom chord of the primary trusses was connected to the bottom chord of the bridging trusses along the length of the primary trusses only on column lines 111, 149, 311, and 349. The connection consisted of double angles 2 by 1 1/2 by 0.25 in. These connection angles were included within the model.

For bridging trusses in the model, a 0.75 in. angle gap was used for trusses with web bar diameters that varied between 0.75 in. and 0.98 in.

Truss Member Cover Plates

In 30 percent of the floor area, truss members were supplemented with cover plates. The members with additional plates included top chords, web members, and most typically bottom chords. Section properties were calculated with SAP *Section Designer*. The primary truss top chords were reinforced with an additional set of double angles at truss end connections. At these locations, the work points for the section were located at the centroid of the composite double angle and concrete slab.

The Laclede shop drawings indicated plates 3/8 in. by 3 in. connecting the bottom chord of the primary truss pairs together at each end and where intersected by a bridging truss. These plates were included in the model.

Viscoelastic Dampers

Viscoelastic dampers were located where the bottom chords of the long span, short span, and bridging trusses intersected the exterior columns. The dampers were defined in Drawing Book D. The dampers resisted static and quasi-static loads (such as gravity loads) at the time of load application. Immediately following load application, the dampers shed load until the stress in the dampers was dissipated. A placeholder element was located in the model at the damper location.

Strap Anchors

Exterior columns not supporting a truss or truss pair were anchored to the floor diaphragm by strap anchors. These strap anchors were connected to the columns by complete penetration welds. The strap anchors were then connected to the slab with shear stud connectors and to the top chords of the trusses by fillet welds. The straps were included in the model and located in the plane of the centroid of the composite top chord. Also, in the model the work points intersected with the centerline of the column and used a rigid link to attach back to the spandrel (see Fig. B–29).

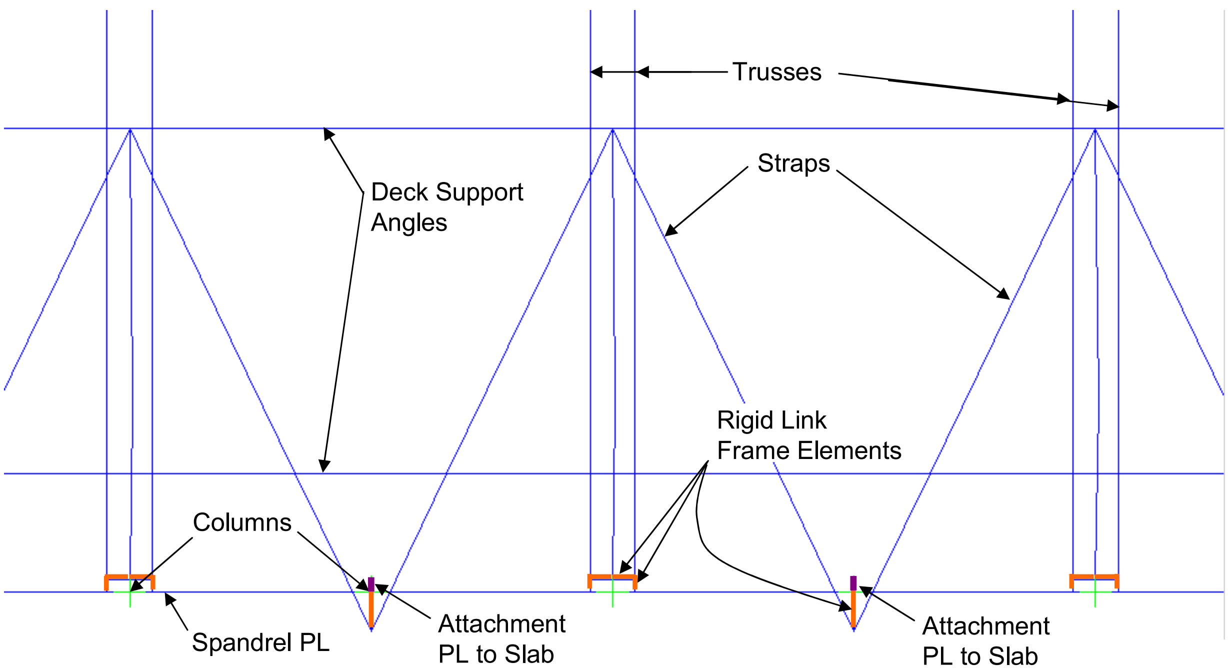


Figure B–29. Strap anchors modeling, slab not shown.

Concrete Slab and Metal Deck

Outside the core, the primary trusses acted compositely with the 4 in. concrete slab on 1 1/2 in. metal deck. In the model, the average depth of the slab plus deck was modeled as 4.35 in. The concrete slab consisted of lightweight concrete with a self-weight of 100 pcf and a design compressive strength, f'_c= 3,000 psi. The concrete modulus of elasticity, E_c, used for modeling is 1,810 ksi, and the calculated modular ratio, $n=E_s/E_c$, is taken as 16, where E_s is the steel modulus of elasticity. These values are consistent with those included within the WTC Structural Design Criteria Book.

Typically, inside the core, the beams acted compositely with a 4 1/2 in. formed concrete slab. The concrete slab consisted of normal weight concrete with a self-weight of 150 pcf and a design compressive strength, f'_c= 3000 psi. The concrete modulus of elasticity, E_c, used for modeling was 3,320 ksi and the calculated n ratio, E_s/E_c, was taken as 8.7.

The floors of the WTC towers had an in-floor electrical distribution system of electrified metal deck and trench headers. The effects of the in-slab trench headers were accommodated by reducing the slab shell element thickness. A 1 ft 8 in. wide shell panel (the typical truss-floor shell mesh size) was reduced in thickness from 4.35 in. to 2.35 in. or 1.35 in. at the trench header locations per drawing SCA–109 (Floor 96A Structural Concrete Floor Plan).

Initial Verification of the 96th Floor Model

Several steps were taken to verify the model input. SAP2000 Version 8 offers a 'shading' option once a model has been built with frame section assignments. This allows the user to view the members as the program has interpreted their input. The shading option was helpful for using section designed shapes, and for verifying the orientation (i.e., local axes) of members. The work was independently reviewed by engineers not associated with the initial model development.

Once the model was completed, checks were performed for gravity loads. All superimposed dead loads and live loads included in the model were based on WTC Design Criteria; self weight is accounted for by SAP2000. To justify the modeling assumptions, several studies were performed to compare stress results to hand calculations for representative composite sections. Hand calculations estimate deflections and member stresses for a simply supported composite truss under gravity loading. For the composite truss sections, the steel stress results were within 4 percent of those calculated by SAP2000 for the long-span truss and 3 percent for the short-span truss. Deflections for the beams and trusses matched hand calculations within 5 to 15 percent.

B.3.3 Typical Beam-Framed Floor Model—Floor 75B

As described in Section B.3.2 for truss-framed floors, the structural drawings were reviewed to identify structural similarities between the beam-framed floors within the expanded impact and fire zones of both towers. It was found that floor 75 of WTC 2 (75B) represents the typical beam-framed floor in the expanded impact zone for WTC 2 (floors 74B to 88B). There were no beam-framed floors within the expanded impact zone of WTC 1.

Floors 75 and 76 of WTC 2, lower and upper mechanical equipment (MER) floors, respectively, were typical of the lower three mechanical equipment floor pairs in both towers (floors 7 and 8, 41 and 42, and 75 and 76 for both WTC 1 and WTC 2). Floor 77 of WTC 2, a lower escalator floor, was a beam-framed floor similar to the lower floor of the mechanical equipment floor pairs, i.e., floor 75B.

Based on the above discussion, floor 75 of WTC 2 was selected as the overall representative beam-framed floor for the expanded impact and zone in both towers and is described in the following sections (see Fig. B–30). An isometric view of the typical beam-framed floor model is illustrated in Fig. B–31. Table B–2 includes a summary of the size of the 75B floor model. The following presents the major structural systems and components of the beam-framed floor model.

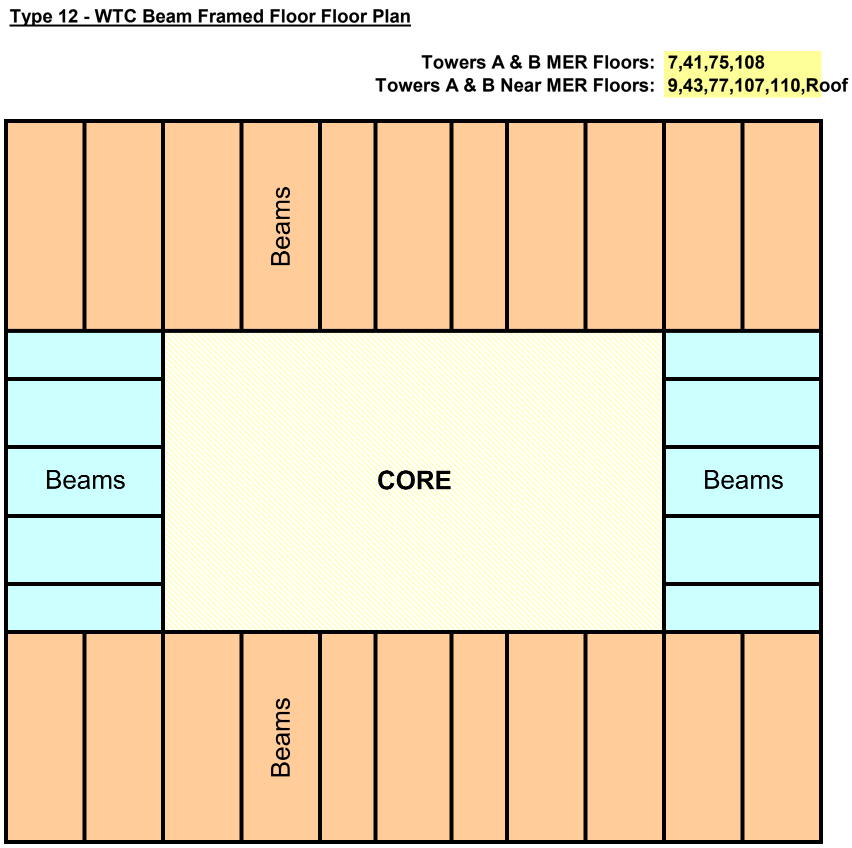


Figure B–30. Typical beam-framed floor arrangement.

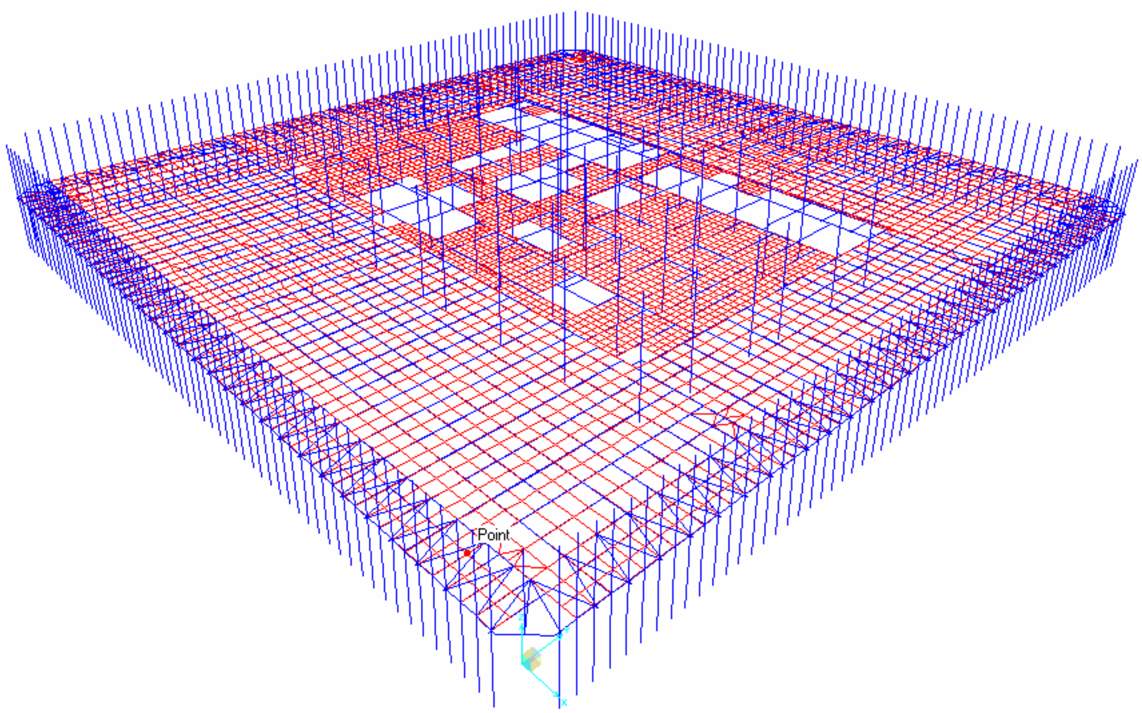


Figure B–31. Typical beam-framed floor model (floor 75B).

Composite Beams

The beams in the model were located at the elevation of the centerline of the concrete slab. The insertion point for the beams was set at the beam top flange, and then the beam was offset down by one-half the thickness of the slab. The beam was rigidly linked with the slab to simulate the composite action. This option provided for accurate estimation of the composite stiffness of the floor.

For beams with cover plates, the properties were calculated by SAP *Section Designer*, and the slab, beam, and reinforcing plates were rigidly linked.

Horizontal Trusses

Exterior columns which did not support a beam were connected to the floor for bracing purposes by horizontal trusses. These exterior horizontal trusses were anchored to the columns with complete joint penetration welds. The horizontal trusses were then connected with shear stud connectors to the slab. The truss angles (typically 4 by 4 by 5/16 in.) were then connected to the top flange of the beams. In the model, the work points intersected with the centerline of the column and used a rigid link to attach back to the spandrel. The truss members were located in the plane of the centroid of the composite top chord (see Fig. B–32).

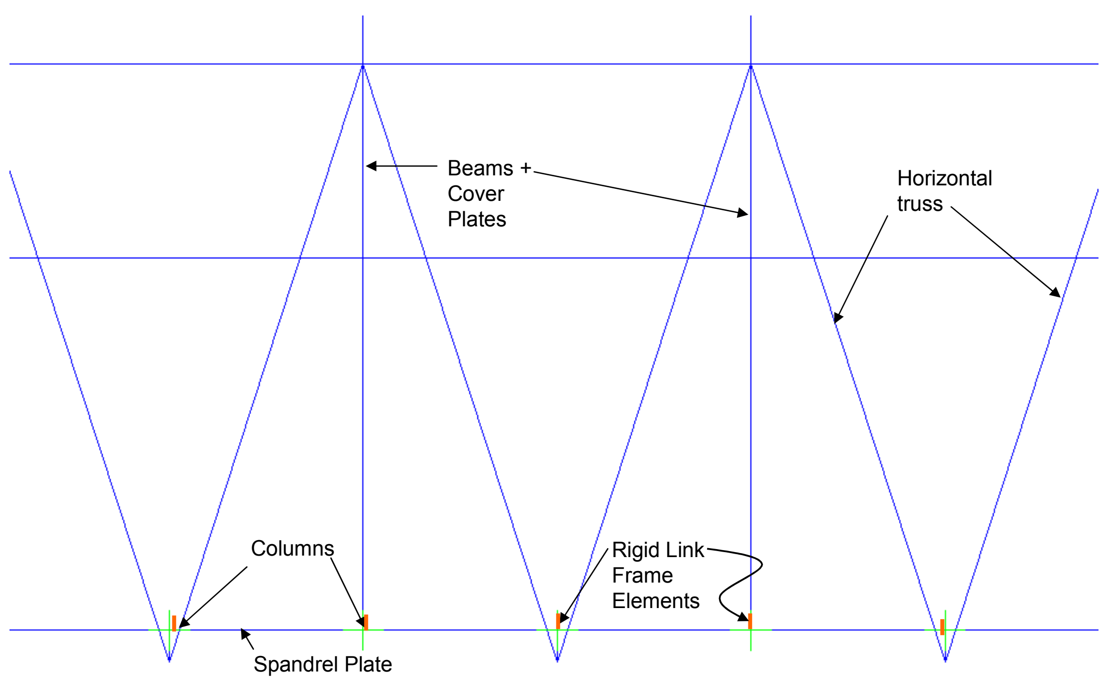


Figure B–32. Horizontal truss modeling, slab not shown.

Concrete Slab and Metal Deck

Outside the core on the mechanical floors, the beams acted compositely with a 5 3/4 in. concrete slab on 1 1/2 in. metal deck. The average cross-sectional depth of the slab in the model was taken as 6.1 in. The concrete slab consisted of normal weight concrete with a self-weight of 150 pcf and a design compressive strength of typically f'_c= 3,000 psi. The concrete modulus of elasticity, E_c, used for modeling is 3,320 ksi and the calculated modular ratio, n, is taken as 8.7.

Typically, inside the core, the beams acted compositely with a 6 in. formed concrete slab. The concrete slab consisted of normal weight concrete with the same properties as concrete outside the core.

The mechanical floors had a 2 in. maximum depth topping slab both inside and outside the core. The topping slab stiffness was not included in the models, but the weight will be accounted for in the baseline analysis.

Viscoelastic Dampers

Viscoelastic dampers were located below the bottom flange of the beams where the beams intersected the exterior columns. Similar to the 96 floor model, a placeholder element was located in the model at the damper location.

Initial Verification of the 75 Floor Model

Similar to the 96 floor model, the 'shading' option in SAP2000 was used to view the members as the program has interpreted their input. The shading option was helpful for using section designed shapes, and for verifying the orientation (i.e., local axes) of members. The work was independently reviewed by engineers not associated with the initial model development.

Once the model was completed, checks were performed for gravity loads. All superimposed dead loads and live loads included in the model are based on WTC Design Criteria; self weight is accounted for by SAP2000. To justify the modeling assumptions, several studies were performed to compare stress results to hand calculations for representative composite sections. Hand calculations estimate deflections and member stresses for a simply supported composite beam under gravity loading. The model yielded accurate steel stress results compared to hand calculations—around 1 percent for both short and long span beams. Where the beams were built-up with reinforcing plates, it was found that SAP *Section Designer* shapes were not calculating the stresses correctly, so instead, separate beam and plate elements drawn over each other were inserted. This method yielded very accurate steel stress results—between 1 percent and 2 percent for both short and long span beams.

B.3.4 Parametric Studies

Modeling techniques employed in the development of the global models of WTC 1 and WTC 2 are consistent with, but often more advanced than, the techniques typically employed in the analysis and design of high-rise buildings. As such, building components were idealized so that overall performance was replicated while appropriately reducing the computational requirements. The following describes the studies undertaken to establish the idealizations used in the models including typical exterior wall panels, exterior corner panels, and flexible floor diaphragms.

Exterior Wall Columns/Spandrel Typical Panels (Floors 9 to 106)

A parametric study of typical three-column, three-spandrel exterior wall panels from the face of the towers (floors 9 to 106) was performed using two modeling methods (see Fig. B–33). The first model was a detailed shell model where each plate of each column or spandrel was specifically modeled, and the second was a simplified frame model. The parametric study assumes that the shell model best represents the as-built panel performance, and therefore, it was used to tune the performance of the frame model which was used throughout the global model (see Section B.3.1). The objectives of the study were to (1) match the axial stiffness of the frame model with the detailed shell model under gravity load and (2) match the inter-story drift of the two models by modifying the rigidity of the column/spandrel intersections in the frame model.

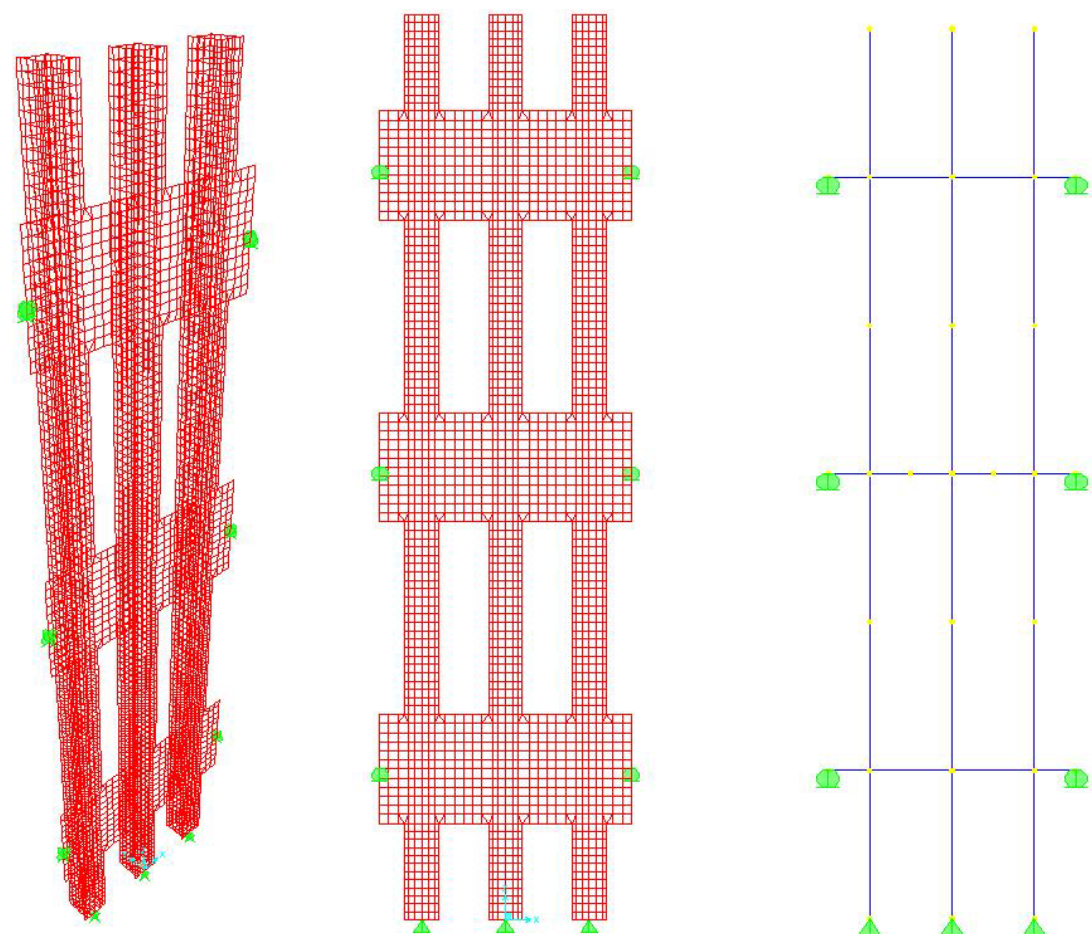


Figure B–33. Shell element and frame models of typical exterior wall panel.

For the axial stiffness of the simplified frame model of the panel versus the detailed shell model, results of loading both models vertically indicated that the shell model was stiffer than the equivalent beam model due to the contribution of the spandrel beams to the columns' axial stiffness. This is due to the rigidity of the spandrel beams and the proximity between the columns. The parametric study on a wide range of panels over the height of the towers showed that the vertical stiffness of the columns in the bottom third of the towers should be increased by a factor in the range of 25 percent to 28 percent, and the columns in the middle and upper thirds of towers should be increased by a factor in the range of 20 percent to 28 percent. Based on these figures, 25 percent increase of axial stiffness of exterior columns was selected as a reasonable representation for the panel vertical stiffness over the height of the towers between floors 9 and 106 (see Section B.3.1).

For studying the lateral deformation of the exterior panels, panel properties were taken from three different areas of the building. These include floors 79 to 82, 53 to 56, and 23 to 26. Internal column stiffeners were included in the shell model. The deformations at points A, B, I, and II (see Fig. B–34) were studied for three different panel locations and their respective spandrel and column thickness. The top most columns were connected via a rigid link and loaded in the plane of the panel and perpendicular to the column with a 100 kip load.

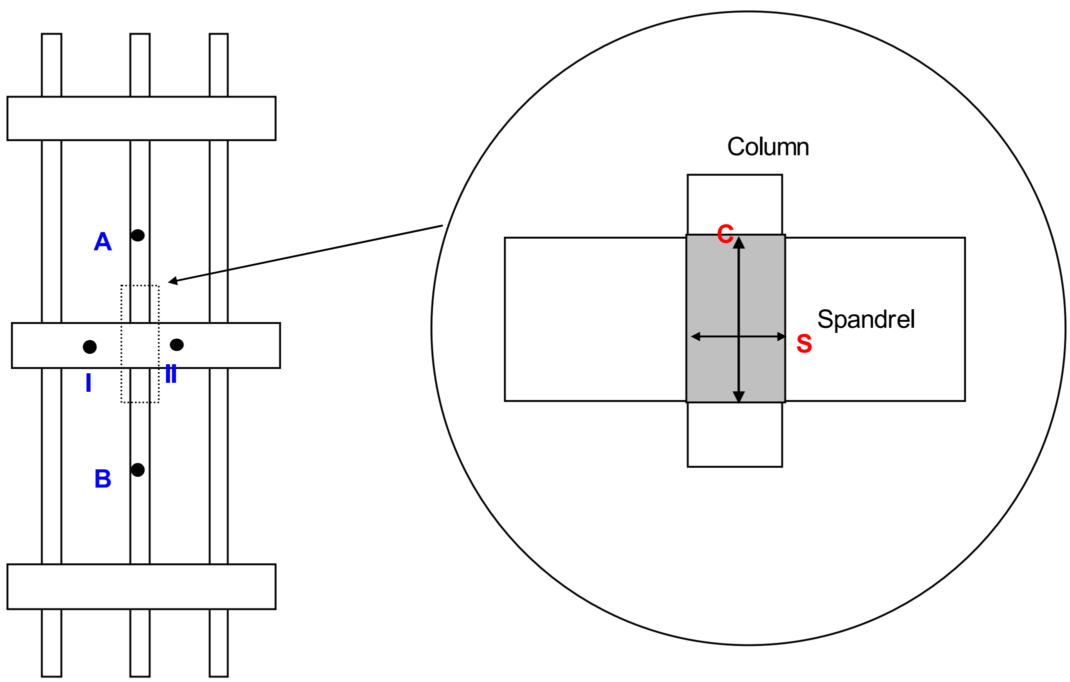


Figure B–34. Column and spandrel rigidity of typical exterior wall panel.

The lateral displacements found for the shell and frame models of typical exterior wall panels with varied column and spandrel intersection rigidities are reported in Table B–3. The study found that 50 percent column rigidity and 100 percent spandrel rigidity in the frame model produced deflection results consistent with the shell model.

Table B–3. Lateral displacement (in.) for the shell and frame models of typical exterior wall panel with varied column and spandrel rigidities.

	Lateral displacement (in)			
	Floor 79-82			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.60	1.04	0.59	0.35
B	0.28	0.52	0.29	0.18
I	0.45	0.78	0.44	0.26
II	0.45	0.78	0.44	0.26
	Floor 53-56			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.26	0.43	0.27	0.18
B	0.12	0.22	0.14	0.11
I	0.19	0.32	0.2	0.15
II	0.19	0.32	0.2	0.15
	Floor 23-26			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.21	0.37	0.21	0.12
B	0.1	0.18	0.1	0.06
I	0.16	0.28	0.16	0.09
II	0.16	0.28	0.16	0.09

Exterior Wall Columns/Spandrel Corner Panels (Floors 9 to 106)

A parametric study was performed of an exterior wall corner panel typical over each corner of the towers from floors 9 to 106. Similar to the exterior typical panels, to account for the contribution of the spandrels into the axial stiffness of the columns, it was found that an area modifier to provide a 25 percent increase in the axial stiffness of the two continuous columns of the corner panels is suitable for modeling the columns' axial stiffness. No modifiers were needed for the 100, 200, 300, and 400 series intermittent columns.

The panel from floor 53 to 56 was selected to be representative with two additional columns attached on either side. The objective of the study was to match the inter-story drift of a detailed shell model and a simplified frame model of the corner panel by modifying the rigidity of the column/spandrel intersections in the frame model. For this parametric study, the panel was straightened to simplify the study and to isolate the behavior of interest (see Fig. B–35). The deformations at points T1, T2, B1, B2, and M2 (Fig. B–36) were studied for representative column and spandrel plate dimensions. The top most columns were connected via a rigid link and loaded in the plane of the panel and perpendicular to the column with a 100 kip load.

The lateral displacements calculated for the shell and frame models of the typical exterior wall corner panel with varied column and spandrel rigidities are reported in Table B–4. The study indicated that 25 percent column rigidity and 50 percent spandrel rigidity in the frame model produced deflection results consistent with the shell model.

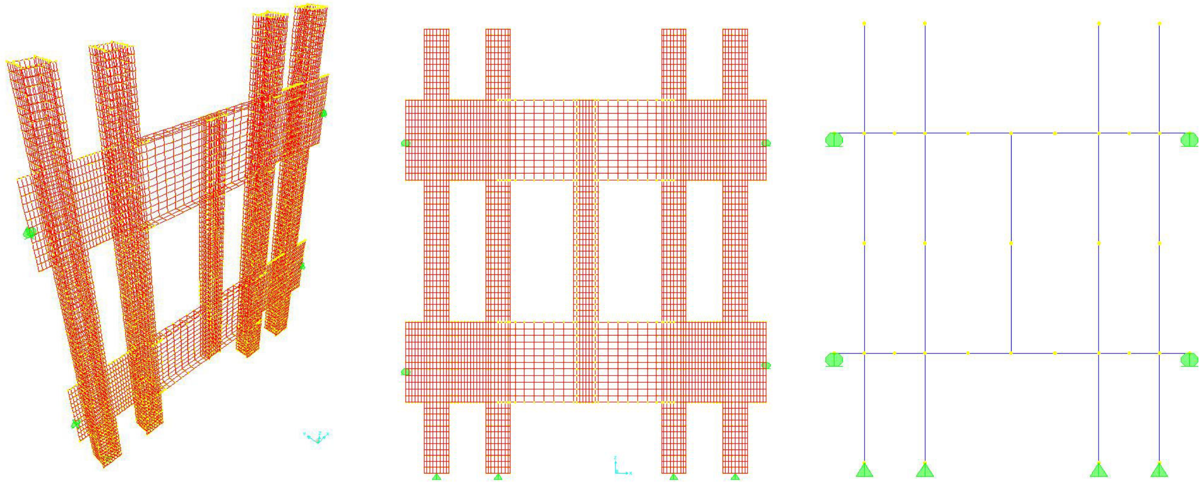


Figure B–35. Shell element and frame models of typical exterior wall corner panel.

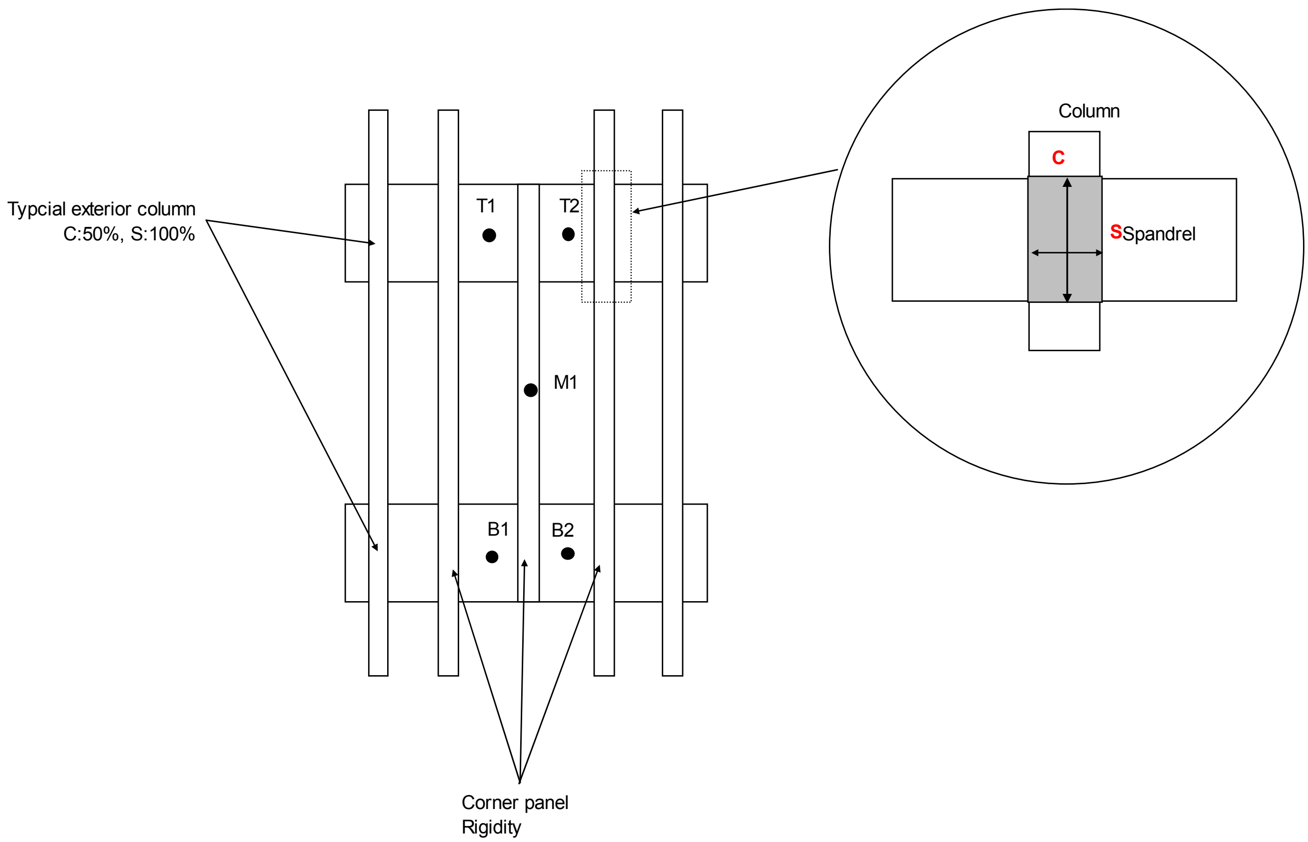


Figure B–36. Column and spandrel rigidity of typical exterior wall corner panel.

Table B–4. Lateral displacement (in.) for the shell and frame models of typical exterior wall corner panel with varied column and spandrel rigidities.

	Floor 53-56			
	Shell model	Corner panel rigidity		
		No rigidity	C:25%, S:50%	C:100%, S:100%
T1	0.227	0.236	0.222	0.152
T2	0.227	0.236	0.222	0.152
M1	0.149	0.154	0.149	0.102
B1	0.084	0.072	0.077	0.053
B2	0.084	0.072	0.077	0.053

As part of the in-house NIST review of the reference structural models (see Section B.4), a detailed shell element model of original corner panel (not straightened) was analyzed under lateral loads to test the accuracy of the simplified frame model with 25 percent column rigidity and 50 percent spandrel rigidity calculated above. Both the detailed and simplified models were loaded as shown in Fig. B–37. The deflections calculated from the frame model were consistent with those estimated from the shell model, indicating that the rigidities estimated using the straight model (Fig. B–35) accurately represent the actual corner panel behavior.

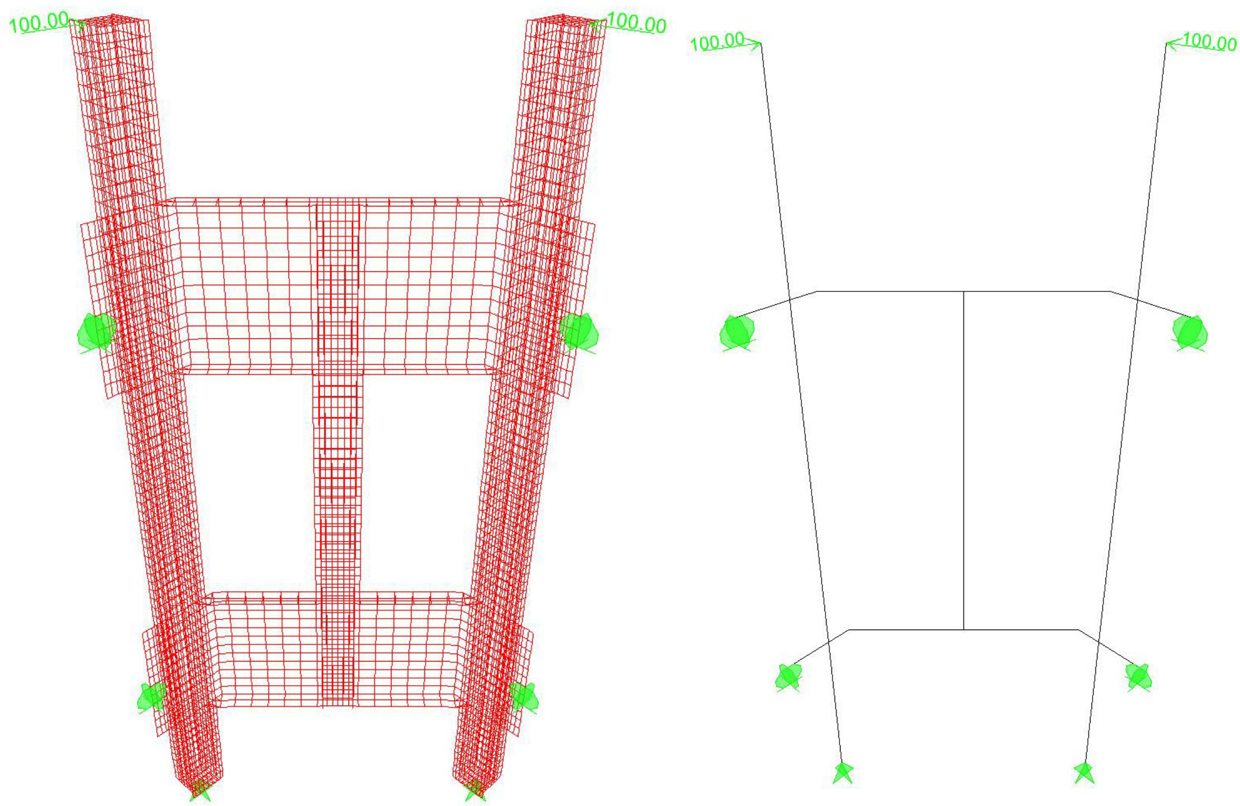


Figure B–37. Detailed and simplified model of the exterior wall corner panel.

Flexible Floor Diaphragm

The floor models developed in Sections B.3.2 and B.3.3 were used to develop the flexible diaphragm stiffness used within the WTC 1 and WTC 2 global models. The in-plane diaphragm stiffness of the detailed floor models was determined and used to arrive at an equivalent shell element floor model. This flexible shell element floor model is then inserted in the global models at specific floors to capture the in-plane flow of forces and deformations. These flexible diaphragms were not used throughout, as the rigid diaphragms in the majority of floors provided for a sufficiently accurate representation of the flow of forces and deformations while keeping manageable the model's computational requirements. In the global models, flexible diaphragms were used at the beam-framed floors 3, 4, 5, 6, 7, 9, 41, 42, 43, 75, 76, 77, 107, 108, 109, 110, and roof of both towers.

Parametric studies were performed to compare the diaphragm stiffness of two different floor models for both the typical truss-framed floor and the beam-framed floor. The typical floor models were compared with the simplified equivalent models that duplicate the representation of the exterior wall columns, exterior wall spandrels, core columns, and their boundary conditions. The floor framing, both inside and outside the core was replaced by shell elements. The material properties of the shell model matched the properties of the concrete floor outside the core in the respective floor model.

The comparative floor models were loaded in the plane of the floors with a lateral load of 180 lb/ft. (equivalent to 15 psf over the 12 ft story height) on both the windward and leeward faces. The column base supports were released for the exterior wall columns along the loaded faces and for all core columns to allow lateral translation only in the direction of loading.

The comparative models were executed to assess the horizontal deflection of the floor on both the windward and leeward sides of the model and for the case where the lateral loads were applied non-concurrently along the 100 face and 200 face of the tower. Both the total horizontal deflection of the slab and the relative displacement between the windward and leeward sides were compared between the models. The shell thickness was modified to match the in-plane stiffness determined by the detailed floor models.

The deformations from the lateral load case using the 96 floor model of WTC 1 are illustrated in Fig. B–38, while Fig. B–39 shows the deformations of the simplified floor model. Fig. B–40 shows the lateral deflection of the north and south sides of the floor model under lateral load applied in the north direction using the detailed and equivalent floor models.

B.4 REVIEW OF THE STRUCTURAL DATABASES AND REFERENCE MODELS OF THE TOWERS

NIST has implemented a rigorous and comprehensive review procedure to mitigate potential conflicts of interest and to ensure the integrity and objectivity of the deliverables of this project, including the structural databases and reference models. The review procedure includes an in-house NIST review as well as a third-party review by the firm of SOM. The following summarizes the results of these reviews for the developed structural databases and reference models.

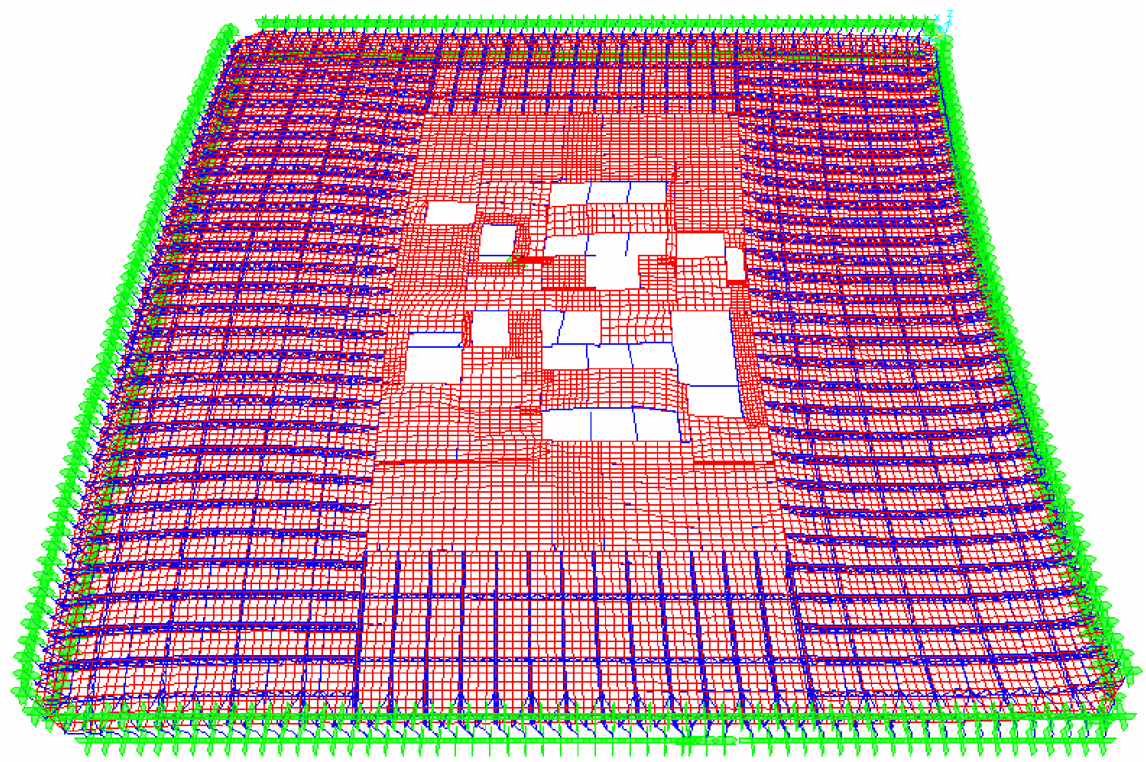


Figure B–38. Deflection of typical truss-framed floor model due to lateral loading (exaggerated scale).

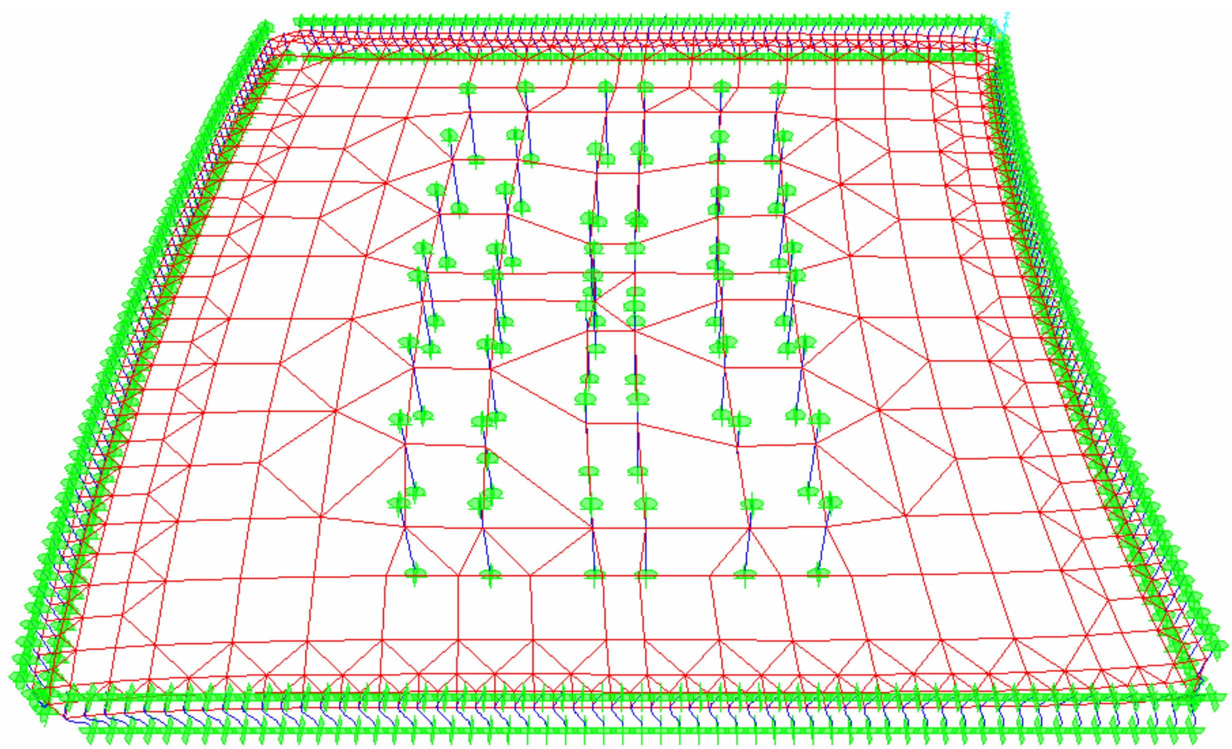


Figure B–39. Deflection of equivalent floor model due to lateral loading (exaggerated scale).

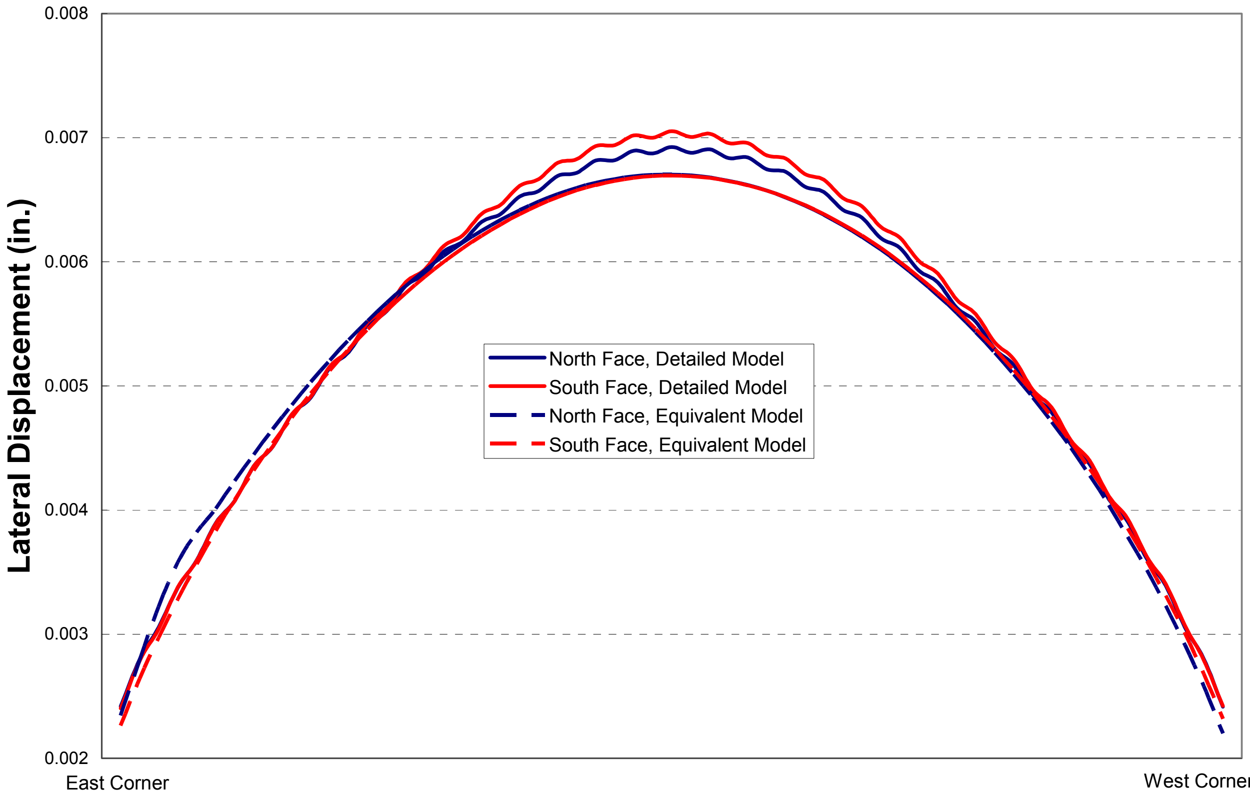


Figure B–40. Deflections of the north and south faces of the floor for the detailed and equivalent floor models.

B.4.1 Review of the Structural Databases

The third-party review by SOM included random checks of the digitized structural databases and cross section property calculations. The review indicated no discrepancies between the developed databases and the original drawing books. Also for cross section property calculations, the review indicated a good agreement (within 1 percent) between the properties in the developed databases and those estimated by SOM. Special attention was given to the calculation of the torsional constant, *J* (see Section B.2.5). It was found that using the software ShapeBuilder, version 3.0 which uses a finite element approach for the calculation of *J* a good agreement was obtained between the *J* values in the developed databases and those estimated by SOM.

The in-house NIST review included the following steps: (1) line-by-line review of all database files, (2) random checks on the developed databases by the project leader, and (3) calculation of all cross section properties and comparison with those in the developed databases. The review indicated minor discrepancies between the developed databases and the original drawing books. For cross section property calculations, good agreement was obtained between the properties in the developed databases and those estimated by NIST. The discrepancies between the developed databases and the original drawing books were reported to LERA, who implemented the changes and modified the databases accordingly. Consequently, the structural databases have been approved by NIST and are being made available for other phases of the NIST investigation.

B.4.2 Review of the Reference Structural Models

The third-party review by SOM included: (1) random checks of the consistency of the developed reference models with the original structural drawings and drawing books, and (2) verification and validation of the models, including reviewing assumptions and level of detail and performing analyses using various loading conditions to test the accuracy of the models. The review indicated that the developed models are consistent with the original design documents. The review indicated that, in general, the modeling assumptions and level of detail in the models are accurate and suitable for the purpose of the project. The SOM review identified two areas where the models need to be modified. The first is the effect of additional vertical stiffness of the exterior wall panels due to the presence of the spandrel beams (see Sections B.3.1 and B.3.4). The second area is the modeling of the connections of the floor slab to the exterior columns of the 75B floor model (Section B.3.3), where this connection appeared to be fixed while the connection should be modeled as pinned.

The in-house NIST review included: (1) checks on the consistency of the developed reference models with the original structural drawings and drawing books, and (2) verification and validation of the models, including reviewing assumptions and level of detail and performing analyses using various loading conditions to test the accuracy of the models. The review indicated minor discrepancies between the developed reference models and the original design documents. Similar to the third-party review, the in-house NIST review identified the proper modeling of the vertical stiffness of the exterior wall panels and the accurate modeling of the floor slab connections to the exterior columns in the 75B floor model as areas that need to be modified in the models.

In addition, NIST conducted a workshop for NIST investigators and contractors to review the reference structural models developed by LERA. The workshop attendees included experts from LERA (two experts); SOM (two experts); Teng and Associates (one expert, outside experts on probable structural collapse); Professor Kasper Willam (outside expert on thermal-structural analysis); Professor David M. Parks (outside expert on computational mechanics for aircraft impact analysis); Applied Research Associates (two experts, contractor on analysis of aircraft impact into the WTC towers) as well as all key investigators from NIST (17 experts). The purpose of the workshop was to discuss the methodology, assumptions, and details of the developed reference models. The minutes of the workshop are being prepared and will be made public. The feedback from the workshop was included in the final review of the models.

The discrepancies between the developed models and the original design documents, as well as the areas identified by both the third-party and in-house review for modification, were reported to LERA, who implemented the changes and modified the models accordingly. Consequently, the reference structural models have been approved by NIST and are being made available for other phases of the NIST investigation.