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A New Twist On Domed Design

The design of a new airport terminal at Ronald Reagan Washington National Airport is evocative of the nearby historic federal buildings

By P. V. Banavalkar, P.E.

When the massive project began a decade ago, the main goals for the new terminal at the Ronald Reagan Washington National Airport were to modernize the airport, enhance passenger amenities, and to improve overall comfort and convenience for travelers. The airport had long battled traffic problems, not to mention problems with clutter and overpopulation.

The new terminal contains 35 new gates with a total area of one million sq.ft. It consists of a main split-level North/South concourse, three finger piers with 31 departure gates, and a four-gate connector to the historic terminal. Additional components are a new roadway system and new parking garages with convenient bridge connections (also from Metro) to the new terminal.

Lateral Load Resisting System

Washington National Airport consists of two distinctive parts. The first part consists of 125'-0" wide terminal with the concourse. The second part comprises three piers adjoining to the terminal that lead to the jet ways. In the terminal building itself, two adjoining bays with plan dimensions of 44'-3" x 44'-3" are crowned by domes with a 16'-0" diameter oculus at their apex, keeping consistency with the architecture of various federal buildings nearby. The pier composes of 28' wide walkway leading to an area of assembly of 92'-0" sq. in plan dimension. The main terminal is isolated from the piers by expansion joints. Furthermore, the terminal along its length composes of segments separated from each other by judiciously placed expansion joints.

The desire for the open space planning and the absence of designated blank walls in the space resulted in providing the rectilinear moment frames for the lateral load resistance of the building. In the terminal along the short direction, the curvilinear roof trusses and rigidly connected floor beams at column lines constitute the moment frames. Along the long direction, to minimize the possible twisting of the terminal building for wind loads, the stiffness of moment frames along each column lines had to be optimized. The tall unbraced columns along the concourse wall necessitated this design refinement. The conventional moment frames provide the lateral resistance in the pier. The in-plane shear stiffness at floor levels is provided by composite concrete slabs; whereas, the roof metal deck ties all structure together between expansion joints at the roof level.

AISC-member Cives Steel Company, Mid-Atlantic Division of Winchester, VA was the fabricator and erector on the project.

HSS Columns

The new terminal has 96 columns consisting of 8"-diameter HSS members surrounded by welded on T-sections. The presence of the HSS members at the middle of the column enhances torsional stiffness of the column. At the same time, it eliminates the necessity of intermediate stiffeners and provides a clean shaft. The column, with 8" diameter pipe at the center, provided an opportunity to drain the roof by a downspout going through the column. Though this solution was not adapted because of the logistic of the construction and possible technical difficulties.

For typical columns with four WT9s welded to 8"-diameter pipe at the center created 28" deep plan dimensions in both directions. The square plan dimensions of the column was architecturally desired. The WT9 sections have flanges 11+ inches wide. The flange of WT6 required for the bottom chord of main roof trusses is approximately 12" wide. An aesthetically pleasing connection was achieved by tapering the flanges of WT6 to match the flanges of WT9.

At the expansion joints, a special configuration of the column with the built-up split channels and a semicircular sector of 12"diameter pipe was provided. The sector of 12"pipe provides torsional resistance to the column. This configuration of the column preserved the original appearance of the column, hiding the expansion joint. The roof trusses on each side of the expansion joint (each with eight in.wide WT flanges) could be supported on these columns without creating disruption in the roof appearance. With these configurations of columns at the expansion joint, a conventional two-column expansion joint was avoided.

At the intersection of the trusses and the column at the interior joints, there are as many as eight members intersecting the column at the top and bottom chord of the trusses. Continuing the 8"-diameter pipe alone beyond the intersection of bottom chord and column created very elegant structural connections. Using an HSS column or cruciform column with wide flange sections alone would have created awkward connections at top and bottom chord junction of trusses and the column.

The depth of main dome trusses along the column lines vary in

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depth from 3'-4" at mid span to 9'-6" at the support. WT6 members are used for the top and bottom chords; whereas, single angles are used as web members. All trusses were shop welded. The four intermediate trusses also utilized similar type of construction.

Each dome at the apex terminates into an oculus with a skylight. The 16'-0" diameter oculus comprises of two W8 x 40 wide flange members vertically connected by W8 x 40 members at the junction radial trusses. The 2'-0" deep oculus acts as a compression ring at the apex of the dome at the same time provides ease of connection to radial trusses. The entire assembly of radial trusses with the oculus was preassembled on the ground and lifted in place by using strong back support.

In the assembly area of the piers, single layered structural baby domes are provided over a square bay of 24' x 24'. Eight-in. diameter pipe columns were used to support the baby domes. The architectural vernacular of using wide flange sections is also maintained in this area.

At the apex of the dome, a 1'-10" diameter prefabricated steel cylinder was used. The cylinder acts as a compression ring at the same time elegantly accommodates bolted connections of eight W6 members. At the base of the dome, the same eight radial ribs of W6 were connected to the eight-in. diameter tubular column. The eight wide flange sections were terminated into triangular prisms. The bolted connections at each end of the radial ribs provided desired construction tolerances and the ease of construction. The gusset plates at the connections were given articulated shape to be consistent with the architectural expressions.

Structural Challenges

Apart from the articulation of the exposed steel structure that is the norm for this quality of structure, the rest of the structure was designed for code prescribed dead, live, snow, wind, and seismic forces. The floor construction of lightweight concrete on metal deck is supported on the composite steel beams. Typical floor beams spanning 44'-3" are 24" deep. The floor was designed not only to carry the imposed loads but was also designed not to be sensitive to floor vibrations. The glass clad terminal building and the piers are designed for the appropriate interstory deflections for wind and seismic loads.

The real structural challenge was to design the air-traffic control tower (ATCT) which is mounted on the top of the terminal and with a shaft diameter of 26'-0" rises to an elevation of 211'-6" above ground level. The location and the height of the ATCT were dictated by the visual field necessary for the desired performance of the air-traffic controllers. The shaft of the tower is a combination of braced and moment frame structure. The required visibility at the upper floors and the operational space requirements necessitated using rectilinear moment frame structure. In the lower trussed portion of the tower, W14 sections are used as columns; whereas, double angles 8 x 6 are used as bracing members. In the upper moment frame W21 members are used as columns; whereas, W24 members are used as beams. Both vertical and lateral load from the tower were to be transferred at the top of the terminal. The conventional wisdom of designing the tower for strength and stiffness for the lateral wind loads does not address the issue of "occupant comfort" of the air traffic controllers. The measure of the occupant comfort is the horizontal acceleration measured in milli-g at the cab level. The preliminary design, though in strict conformance with the guidelines of the FAA when tested in the wind tunnel at University of Western Ontario, showed wind induced acceleration in the range of 70 to 90 milli-g for the inherent structural damping of 0.5% of the tower. This magnitude of horizontal acceleration is intolerable to the air traffic controllers impairing their ability to perform their duties.

Drawing from experience with performance of tall buildings, an operational criteria for the tower under wind was selected. This required that for the 10 year return period, peak resultant acceleration at the cab level be kept below 17 milli-g's. Based upon the fact that the dynamic response of the structure in inversely proportional to the effective mass of the structure and to the square root of the available damping following steps were taken:

- Improve effective mass of the tower.
- Thickening floor slabs at upper floors within the architectural limits increased the physical weight of the tower.
- Eliminating conventional plate girder type transfer girders at its base and mounting the tower on a 10' deep pyramidal truss directly supported on the columns modified the fundamental mode shapes of vibration of the tower.

The pyramidal truss consists of W14 members used as chord members with double angles 8 x 6 used as bracing members. The pyramidal truss practically eliminated the base rotation of the tower and thereby significantly improving the effective mass of the tower. The effective mass of the tower with the added mass of the terminal due to its participation in vibration was determined by a detailed 3-D computer model analysis.

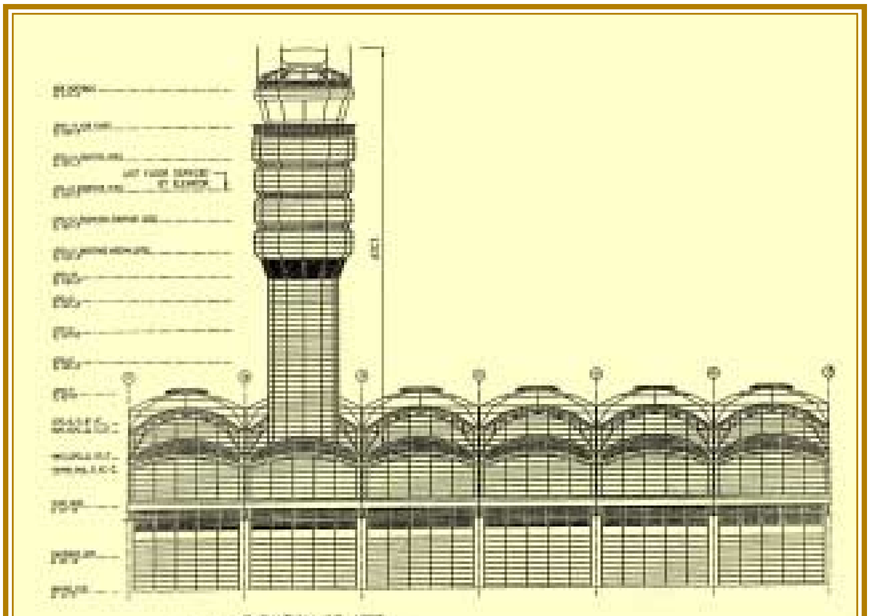
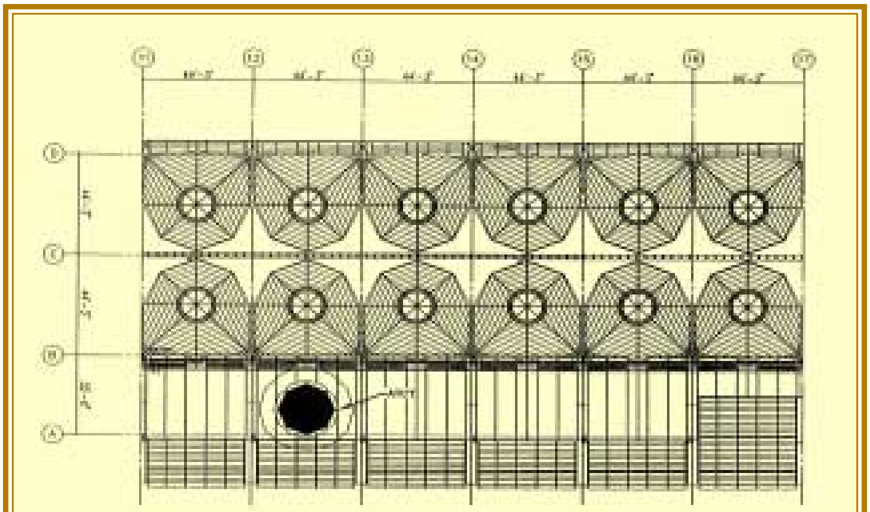


FIG. #1 ELEVATION OF ATCT



PLAN LAYOUT OF ATCT & TERMINAL BUILDING

Added Damping

In spite of the positive measures described above, the damping of the structure still needed to increase to achieve the criteria for horizontal wind induced acceleration at the cab level. In addition to inherent structural damping of approximately 0.5% of the tower, an additional damping of 3% was needed to reach the goal of 17 milli-g acceleration for a return period of 10 years.

After studying different types of damping devices, a tuned mass damper weighing 29,900 lbs. to be installed at elevation 210'-9" was chosen. This is the first such tuned mass damper installation in ATCT in North America. The technique has been successfully adapted in Japan where the structural steel framed towers are a common norm because of the seismic design requirements.

The tuned mass damper, installed in ATCT, was tested for its performance by physically shaking the tower with a custom designed shaker. In order not to disrupt the operation of the airport, the testing of the tuned mass damper was done at night. The damper provided additional 3% damping as desired. The

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damper is continuously monitored at University of Western Ontario by a modem. Records measured during the high winds spurred by Hurricane Floyd this year further substantiated the successful operation of the damper in reducing the horizontal acceleration at the cab level.

Concourse Glass Wall

Approximately 56'-0" vertical span of glass wall is supported by vertical pipe trusses spaced at 8'-0" o.c. The 1'-9" deep pipe trusses with 5" diameter pipes are laced together by batten plates. The trusses at the base are connected to the concourse level by 1 ¾ diameter headed pins.

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