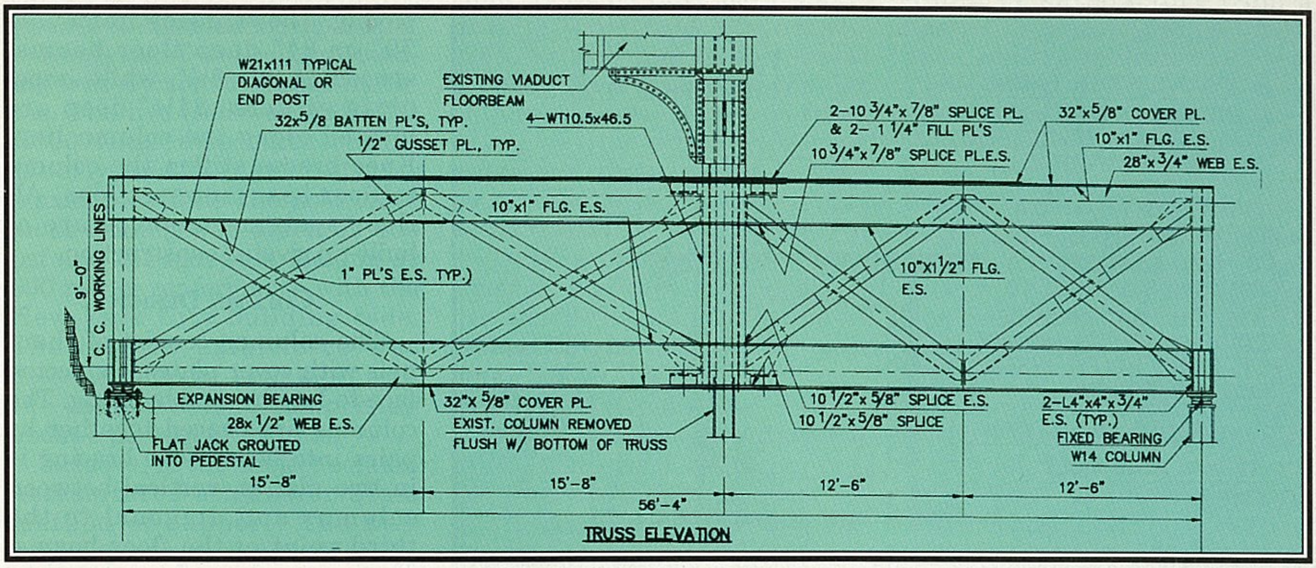

TRANSFER TRUSS SUPPORTS RENOVATION

Large plate girders proved impractical due to the tight site constraints

By Michael A. Lahti, P.E.

AS COULD BE EXPECTED, TRAFFIC PATTERNS HAVE CHANGED SUBSTANTIALLY since New York's Penn Station first opened in the early 1960s. As a result, the Long Island Rail Road—which accounts for two thirds of the station's traffic—is currently undergoing an extensive improvement program. Included in this program is a major track realignment in order to lengthen a specific passenger platform (see "Extended Platform," MSC, February 1996). As a result of this work, 11 columns were removed.

One of the replaced columns was part of a viaduct carrying West 33rd Street between 8th and 9th Avenues, north of the James A. Farley Post Office. The viaduct framing also carries a portion of the "moat", a plaza below street level between the exterior wall of the JAF Post Office and the parapet adjacent to the 33rd Street sidewalk.

The 33rd Street viaduct is a multi-span plate girder structure with a typical span length of 41'-4½". The north bearings are supported by I-beam grillages set into the top of the rock cut bounding Penn Station. The opposite end, 42'-4" to the south, is carried by a line of columns. The deck consists of concrete jack arches on 24" stringers, with earth fill and bituminous

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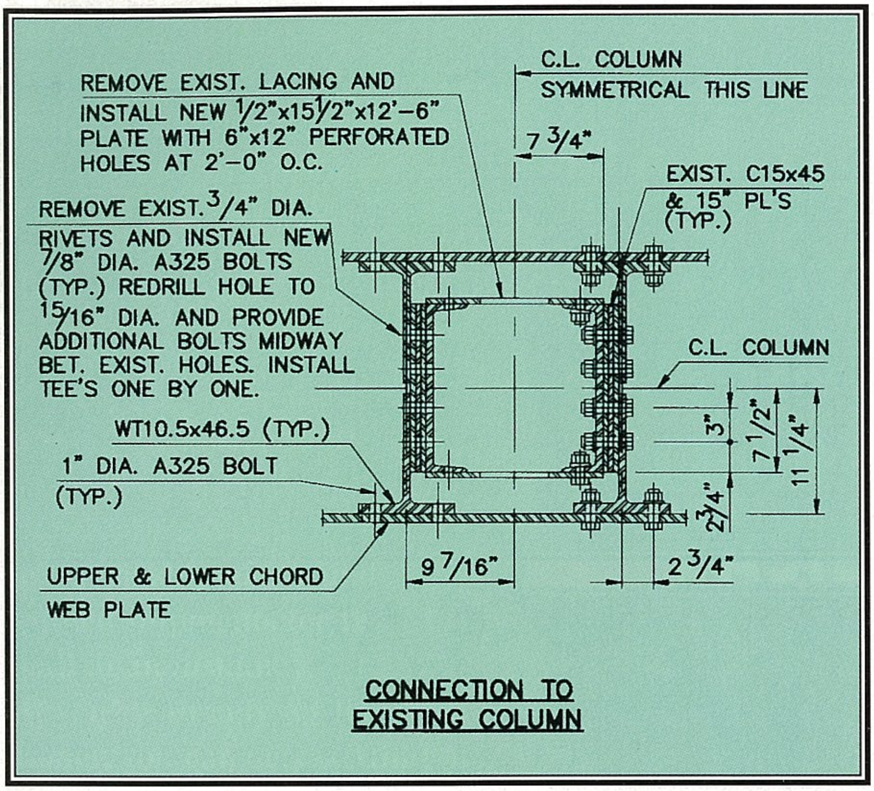

CONNECTION TO EXISTING COLUMN

paving. The stringers frame into 79" or 84" deep floor beams, spaced at 20'- 8½", while steel plate girders 81½"-deep are located along the column line. Knee braces stiffen the column to floor beam intersections. All floor system members are of built-up riveted construction.

Column Design

The columns consist of channels with cover plates, connected face-to-face by double lacing. The columns are braced together by pairs into bents. The bracing is in two planes, vertical between columns and diagonal to the third point of the floor beams. The bracing is configured as double X frames, with numerous secondary members, and is of substantial construction. The post office moat framing, consisting of 12" rolled beam stringers with a 20'-0" span and 18" I-beam girders, is supported by brackets riveted to the columns adjacent to the knee brace. Footings are granite blocks founded on rock.

Analysis of the column loading was by conventional methods. The live load was AASHTO HS20 trucks, distributed longitudinally and transversely for maximum effect. The post office moat loading was taken as 100 psf and the sidewalk loading as 600 psf, in conformance with the New York City building code. The column axial force was determined to be 1140 kips, with the vehicular load comprising about 5 percent of the total.

Removal of the subject column necessitated a transfer beam method of support. The new track layout was compatible with an orientation perpendicular to the viaduct, resulting in a span length of 56'-4", without interference with existing foundations. Alignment parallel to the viaduct would have required placing one of the new supports directly against an adjacent viaduct column. It would have been necessary to strengthen the column for a doubled loading plus transmitting the forces eccentrically into the footing. Replacement of the existing granite footing would have been needed to keep foundation pressures within reasonable limits.

To eliminate need for any shoring, the replacement framing would require installation in halves about the existing column, above the train clearance envelope. Any temporary shoring would have seriously interfered with a yard track needed to provide access for the concurrent track realignment and other ongoing work.

Trusses Prove More Practical

Both plate girders and trusses were considered for main members. Preliminary computations indicated that twin 8' plate girders, weighing about 30 kips each, would be satisfactory from a design standpoint. A constructibility review, however, showed that delivery and erection of these large girders within the confines of the station would be impractical. There was insufficient headroom or turning circle available for efficient crane operation. A truss configuration provided a more workable solution because the individual components weighed less than 10 kips and could be handled without heavy equipment. The overall increase in steel weight and higher fabrication costs were offset by savings in construction convenience.

Different truss styles were then investigated. The final configuration had parallel chords at 9'-0" spacing, with double diagonals and vertical end posts. Intermediate verticals, except at the original column, were unnecessary. Large primary bending

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moments (in excess of 1200 foot-kips) were present in the chords, making analysis of the truss as a rigid frame necessary. Design as an ideal truss would have been inappropriate. The analysis also indicated that one end bearing had to be free to expand, as restraining both ends would have induced a horizontal reaction (larger than 900 kips) upon loading that was sufficient to shear the anchor bolts.

The upper and lower chords consisted of welded A572 Gr.50 channels 28" deep with bolted cover plates and battens. Channels were continuous, but the cover plates terminated at the column connection and were spliced by additional plating. The two center panels of the upper chord had a second cover plate. In addition to a combined stress analysis, the b/t ratio of all plating was verified for compliance with AASHTO specifications.

The lower chord and tension diagonals were designed as fracture critical non-redundant members. The stress range in the Category B flange to web welds was well below the allowable fatigue range.

The upper portion of the existing column was incorporated into the truss as a vertical member. The lacing bars were replaced by perforated plates and four WT10.5 sections, installed to align the column with the truss channel spacing, and to serve as a connecting diaphragm. Four tee sections were used, instead of a 21"-wide flange beam, to facilitate connection without concern about delamination of the existing built-up column under its imposed dead load.

The diagonal members and end posts were W21x111 rolled beams, also of A572 Gr.50 steel. Gusset plates were compact, but could not be entirely eliminated. All connections were 1" diameter A325 bolts and were designed as eccentric bolt groups.

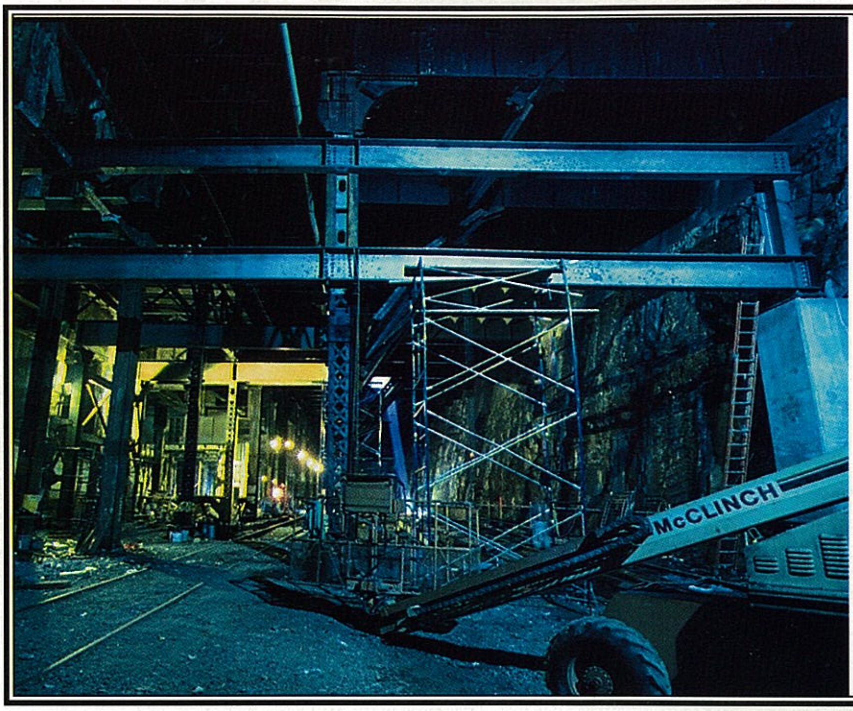
The south end bearing was supported by a W14 column, which was braced into a bent with three other W14 replacement columns. These three columns re-supported JAF Post Office members also displaced by the new track alignment. The north end was placed on a concrete column cast against the rock wall of the station. This column was socketed into sound rock below track level to avoid jointed and seamed rock present on the north wall. The bearings were conventional curved steel shoes, with the north end free to expand.

Fabrication Considerations

Part of the steel fabrication included welding the built-up channels. A pair of channels, top or bottom, was assembled as a box, the full penetration welds applied and the box then split into halves. Welds were visually inspected and tested by the magnetic particle method. The channels were checked for compliance with a camber and sweep tolerance of L/500.

The entire truss was shop assembled on its side to verify alignment of the bolted connections. Lack of fit, particularly in the chord to web connections, would have violated the specifications, which did not permit field reaming of misaligned holes.

The work zone was within active yard tracks on the north side of the station. It was fenced off from revenue service and protected by railroad flagmen while work was in progress. The erector was freed of many of the usual restrictions, such as only working during off-peak hours. Normal productivity was generally possible.



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The structural steel was delivered by truck and lowered to track level from 33rd Street by use of a small crane. Access was through a 20 ft. by 4 ft. hole in the south sidewalk. The members were placed on rail mounted dollies and rolled to the work area.

Steel Erection

Erection of the truss was accomplished mainly with hand tools. The heavy hoisting, upper and lower chord channels, was done using two eight-ton chain falls. A diesel powered manlift was also utilized, both for access and to lift smaller components.

The connection to the existing column had to be installed in a specific order to avoid the need for shoring. The minimum number of rivets were removed at each step of the procedure. First, the lacing bars were replaced by perforated plates, bolted up one side at a time. Then, the structural tees were connected, one corner at a time.

The truss assembly was carried out in a progressive manner and did not require any falsework. The lower chord channels were bolted to the column structural tees and the ends blocked into position, the end posts were placed and the upper chord channels connected. Guy cables held these members in alignment while the remainder of the material was bolted in, piece by piece. The primary lateral members were not removed until the truss was sufficiently complete to provide bracing for the existing column. The new bracing was then connected. Field welding was permitted for connection to existing steel, but only to secondary components, not to tension flanges or cover plating.

The jacking scheme proposed in the plans was modified at the contractor's request to use equipment already available. The original scheme was to fit brackets and jacks to the existing column, cut the column, lift the structure and then shim the truss bearings into place. The contractor's method involved lifting from the ends of the truss. Both ends would be jacked in proportion to their calculated reactions, maintaining a level lift, until the existing column top was $\frac{1}{16}$" to $\frac{1}{8}$" above its present elevation. The column would then be cut and the final elevation of the truss adjusted.

The jacking arrangement was different at each end. The south end, supported by a W14 column, was lifted by means of brackets near the base. Steel wedges were driven to fix the baseplate in position. The footing was constructed with sufficient width to accommodate two 565 ton jacks. The jacks and brackets were previously used for two adjacent column replacements.

The north end was lifted by a flat jack installed below the expansion bearing. This jack was pumped with an epoxy grout instead of hydraulic fluid. The jack was subsequently grouted into and remained a part of the pedestal.

The measured loads, determined from the gauge pressures, were close to the computed values. The entire jacking operation, including cutting of the original column, required about two hours.

The truss erection, including removal of existing members and installation of bracing, but exclusive of concrete work, took slightly more than two weeks duration. The crew consisted of five ironworkers and a foreman.

General contractor on the project was A.J. Pegno Construction Corp. of College Point, NY, erector was Budco Enterprises, Inc. of Middle Island, NY, and structural engineer was Lichtenstein Engineering Associates of New York.

Michael A. Lahti, P.E., is an associate with the Lichtenstein Engineering Associates, P.C., a consulting engineering firm with offices in New York City, New Jersey, Pennsylvania, Massachusetts, Connecticut, Ohio and Florida.