Vertical Integration at Vanderbilt University

Post-tensioned transfer truss system allows medical center to expand above existing auditorium

BY OTTO J. SCHWARZ

As part of an initiative to increase its ranking among medical research institutions, the Vanderbilt University Medical Center (VUMC) has undertaken a major expansion project. A key component of this expansion is the VUMC Medical Research Building IV that will house research programs and staff in a state-of-the-art laboratory facility with over 400,000 ft^2 (37,000 m^2) of space. A lack of green-field sites available on campus that could hold such a facility, however, produced an engineering and construction challenge that required an unorthodox solution.

WORKING AROUND CONSTRAINTS

Planning for the facility included an extensive study of potential building sites within the VUMC campus, located just south of downtown Nashville, TN. Because no green-field sites with access to campus services and utilities were available within the research campus, the search was extended to currently occupied sites. The best location was found to be the site shown in Fig. 1, containing Langford Auditorium and Light Hall; the auditorium, however, was booked for the foreseeable future and had the only 1200-seat auditorium on campus—it could not be closed.

Phased construction

In early 2003, VUMC authorized its design team to proceed with an ambitious plan to capture the air space above the auditorium and the adjacent building. The project was partitioned into two distinct design and construction phases. Phase I entailed the addition of four floors, totaling 124,000 ft^2 (12,000 m^2), to Light Hall, a nine-story building next to Langford Auditorium (Fig. 1). Light Hall's existing cast-in-place concrete structure was used to support new columns and two-way, post-tensioned

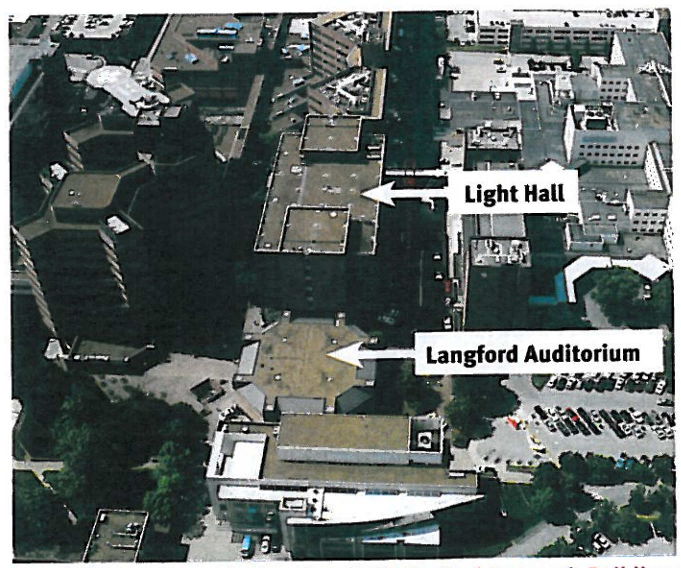

Fig. 1: The optimum site for the VUMC Medical Research Building IV was occupied by Light Hall, constructed in 1975, and Langford Auditorium, constructed in 1977 *(Photo courtesy of Aerial Innovations of TN, Inc.)*

(PT) concrete floor and roof slabs. Phase II entailed the construction of a new 14-story facility totaling 275,000 ft^2 (25,000 m^2), with most of this space provided on the 11 floors located above the auditorium. Analyses had shown that the existing framing, consisting of wide flange columns supporting a concrete slab on metal deck roof over 72 in. (1830 mm) deep joists, had little additional capacity to support gravity loads—significantly less than that imposed by 11 cast-in-place concrete floor slabs. Strengthening of the foundations and existing vertical support system, or addition of new interior columns, would have required disruption of the events scheduled for the auditorium. Therefore, the new structure had to incorporate a transfer and support system to span the 135 ft (41 m) wide, 45 ft (13.7 m) tall auditorium.

Transfer system

The design of the transfer structure was driven by numerous constraints:

- Construction could not interrupt operation of the auditorium. Underground utilities and access tunnels located on the east and west sides of the auditorium had to remain in service;
- The elevations of floors constructed over the auditorium had to match those of the existing and new floors in Light Hall to allow spaces developed in Phases I and II to be contiguous. This constraint was quite significant, as the space between the auditorium roof and the seventh floor of Light Hall limited the floor-to-floor height at the transfer level to only 22 ft 10 in. (6.95 m);
- The transfer structure was required to meet vibration and deflection criteria set by the owner, including a net deflection limit of 1 in. (25 mm) across the 135 ft (41 m) wide auditorium, to protect sensitive equipment housed in the upper floors of the facility; and
- The transfer structure had to serve as a mechanical equipment level, providing space and support for the extensive systems needed by the laboratory equipment on the upper levels. The structure therefore had to accommodate large ducts up to 72 in. (1.8 m) in diameter in orthogonal directions, while allowing access for installation and maintenance of air handling units and filter rooms.

These requirements, as well as the limited crane and site access in the middle of a busy campus, led to the selection of a four-truss, PT, cast-in-place concrete transfer structure. The plan for the trusses is shown in Fig. 2. An elevation of the truss along truss line 2 is shown in Fig. 3. In addition to providing the required stiffness and strength, the truss system provided multiple web openings for mechanical systems, fire resistance, and staged tensioning that allowed camber adjustment as the subsequent levels were constructed.

As difficult as it was to find a location for the building, it proved almost as challenging to find support locations for the transfer structure. Because columns could obviously not pass down through the roof and into the middle of the auditorium, the supports for the new structure were limited to locations near the perimeter of the existing building. The support locations and the new foundations, however, also had to miss the existing utility and access tunnels that served the adjacent research facilities and could not be shut down or disturbed.

After careful consideration, the eight major support points shown in Fig. 2 were finally established for the trusses. Along the west side of the auditorium, 4 and 5 ft (1.22 and 1.52 m) square columns and the northeast and southeast corners of the east elevator shaft were chosen as support points. Along the east side of the auditorium, wall and column elements were extended through the elevated framed terrace to support north and south exterior trusses. Some existing ductwork was also relocated from two existing mechanical chases, and the chases were converted into 6 x 10 ft (1.8 x 3 m) reinforced concrete columns to provide support for the two interior trusses. The chases didn't align with the columns on the west side of the auditorium, so a 6 x 12 ft 6 in. (1.8 x 3.8 m) PT transfer girder was provided between these columns (Fig. 2 and 3).

ANALYSIS AND DESIGN

Analyses of the gravity and lateral system of the structure, including the PT truss system, were conducted using three-dimensional models. The model of the completed structure was subjected to 90 mph (145 km/h) wind loads and designed for Category A seismic loads, as defined by the 2000 International Building Code.[1]

The lateral support system for the PT one-way beam and slab structure above the transfer level consisted of a series of reinforced concrete structural walls. The stiffness of the PT transfer structure interacted with these walls to form a hybrid truss to wall moment frame, providing relative fixity of the lateral system between the fifth and seventh floors and greatly stiffening the structure against lateral forces.

Staged analyses

A total of five models of the structure were used to reflect the completion of construction up to and including floor slabs at Levels 7, 8, 11, 13, and 16. A sixth model, which included only the truss system and support elements, was used to evaluate the trusses without the stiffness contribution brought about by Vierendeel action of the supported frames. Each of these models was analyzed with the applicable gravity and lateral loads to determine the forces on the transfer structure elements at each stage of construction. PT forces affecting truss and transfer girder elements were included in each of the

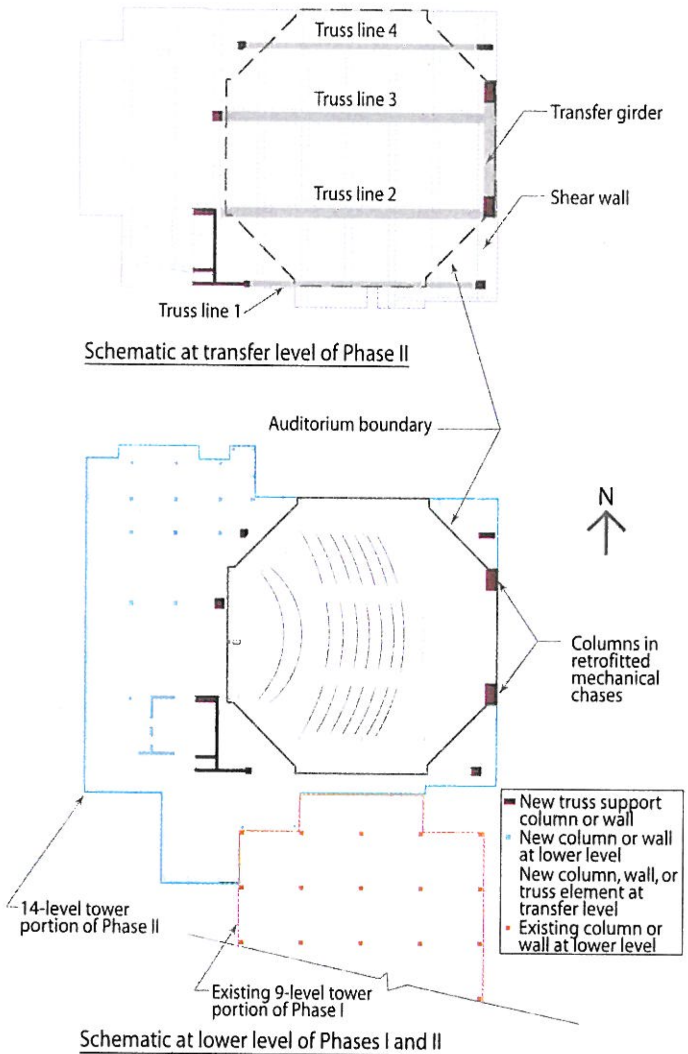

Fig. 2: Plans at ground floor and at transfer truss bottom chord (Level 5)

applicable models. Both the initial and final (after losses) conditions at each stage of tendon stressing were included. Additionally, long-term creep deflections were modeled using thermal loads.

Design

Calculated member forces were compiled in a spreadsheet, and design was conducted using a basic three-step iterative process to satisfy requirements indicated in ACI 318-99.[2] First, using the allowable flexural stress limits, the required reinforcing and PT force were calculated for each chord and web member. The PT force was then checked and adjusted to meet minimum factors of safety against net section cracking and net section decompression. Finally, the capacity of each member was analyzed based on strain compatibility of the bonded PT tendons and reinforcing bars. If required, the member was reproportioned to meet strength requirements at the critical sections. Any change in net PT force or required member cross section dimensions triggered another iteration of the analysis and design process.

During the design process, it quickly became apparent that congestion at the truss joints, such as shown in Fig. 4, would present one of the greatest challenges for both detailing and construction. Close coordination between Carpenter Wright Engineers (CWE) and VSL, the post-tensioning supplier and detailer, was required to develop layouts of the reinforcing steel, PT ducts, and anchorage devices that would satisfy constructibility and consolidation requirements. Janssen and Spaans Engineering was contracted by VSL to provide engineering services for the anchorage zone design and detailing of the reinforcement and PT strand.

To prevent joint failure and provide anchorage at the truss joints, development of conventional reinforcement (often No. 11 and No. 14 [No. 36 and No. 43] bars) was achieved using mechanical couplers bearing against structural steel plate washers. For No. 14 (No. 43) bars,

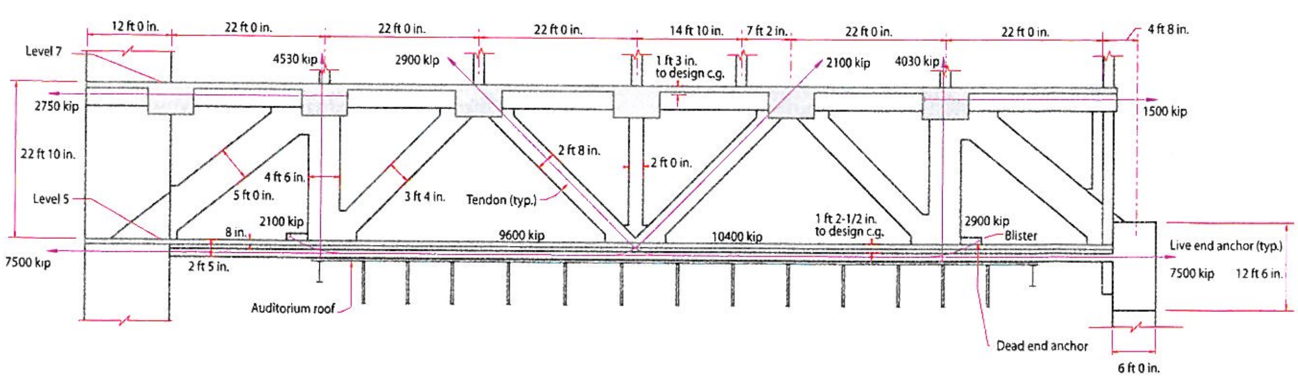

Fig. 3: Elevation of truss line 2 (1 ft = 0.305 m; 1 in. = 25.4 mm; 1 kip = 4.45 kN)

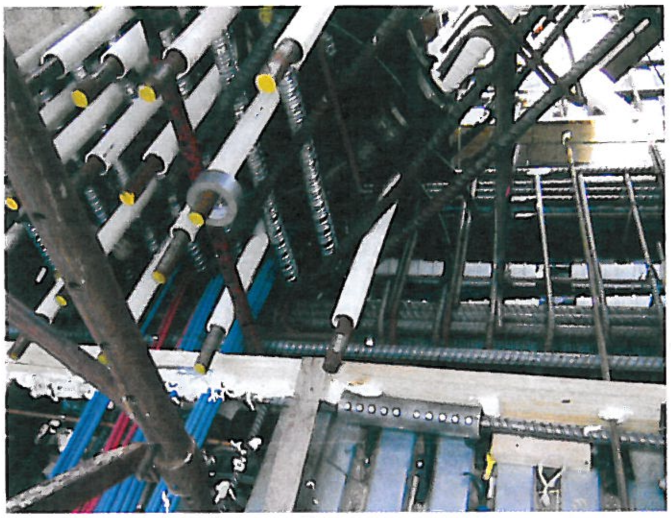

Fig. 4: Accurate detailing and constructibility reviews were essential. Shown in this photo of a joint at the center of a bottom chord are longitudinal reinforcing bars, transverse reinforcement, unbonded PT strand for the transverse floor beams (red and blue), bonded PT ducts (white and silver), high-strength PT bars to resist general zone bursting forces (yellow ends), and a cheek plate

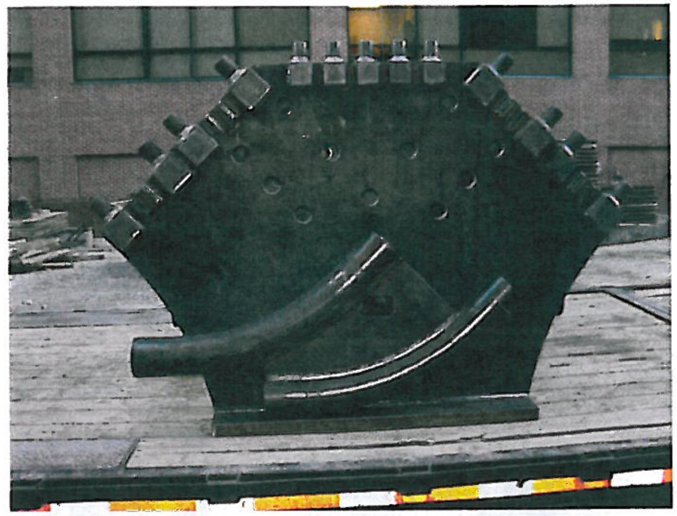

Fig. 6: Steel cheek plates were used to simplify the anchorage and harping of the post-tensioning reinforcement. The tubes near the base were used to deviate strands from the bottom chord into one of the truss diagonals. The threaded connectors along the top and sides were used to attach PT bars up to 2-1/2 in. (64 mm) diameter to the plate. Holes in the plates allowed the passage of horizontal high-strength confining bars

Fig. 5: View of the center of a nearly complete transfer truss

Fig. 7: HSS trusses spanning over 132 ft (40 m) formed an internal skeleton that was used to support the formwork while the concrete transfer trusses were constructed

these plates were 6 in. (150 mm) square and 2-1/4 in. (57 mm) thick. The 2-1/2 in. (64 mm) diameter PT bars used in the vertical tension elements of the trusses carried 4500 kips (20,000 kN) of final effective force. Anchorage for each of the two sets of five PT bars in these members was provided by steel anchor blocks at the ends of the web member that were 8 in. (200 mm) wide, 4 ft 5 in. (1.35 m) long, and 5 in. (125 mm) thick.

Bottom chord PT forces up to 10,400 kips (46,300 kN) were required at interior truss panels to resist the tension forces. Based on previous experience with similar construction, truss geometry was developed to minimize joint locations where the PT would need to be anchored in more than one direction. External anchor blocks (blisters) were provided at the first interior panel point at each end of the truss to provide room for intermediate tendon stressing and reduce the total PT force anchored at any one location. Even with this reduction, end-of-truss anchorage zones transferred 7500 kip (33,400 kN) of final effective tendon force.

Harping of the multistrand ducts was eliminated except at the center bottom chord joint where three web members intersected (Fig. 3 and 5). To contain the radial and bursting forces within the joint and to anchor the

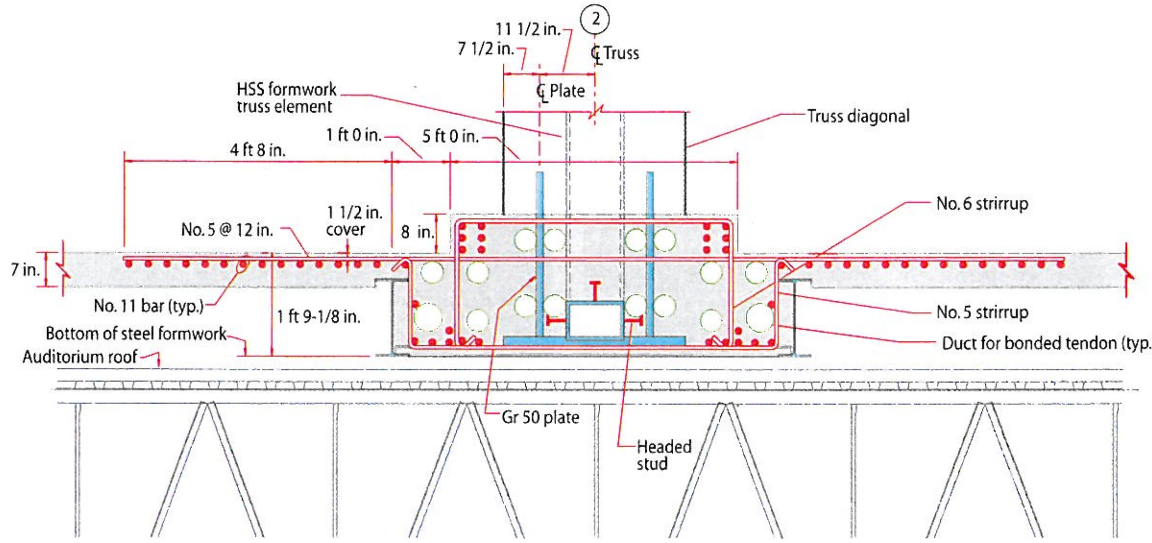

Fig. 8: Section through the bottom chord of a truss at the location of a cheek plate. The HSS sections in the support truss are also shown (1 in. = 25.4 mm)

reinforcing steel to the base of the joint in a manner that would minimize congestion, structural steel cheek plates (Fig. 6) were designed and fabricated. Holes were provided through the two vertical plate assemblies to allow passage of lateral PT bars (Fig. 4) that clamped the joint together and resisted general zone bursting forces.

CONSTRUCTION

To overcome the challenge of constructing the transfer level trusses without damaging or disturbing the auditorium below, John Carpenter of CWE devised a unique solution. A structural steel platform supported by grouted and PT hollow structural steel (HSS) trusses was erected to provide a formwork skeleton for the PT cast-in-place concrete truss. Each HSS truss was designed to support the self-weight of the concrete truss to be cast around it. One of these trusses is shown in Fig. 7 as it is being lowered through one of the cheek plates. The truss also appears in the bottom chord section shown in Fig. 8.

After construction of the concrete support elements to the elevation of the future fifth floor, a relatively lightweight structural steel formwork system was laid out over the roof of the existing auditorium. At the main two interior truss lines over the auditorium (truss lines 2 and 3 in Fig. 2), HSS trusses spanning 132 ft (40.2 m) were erected between the supports on the east and west sides of the building. The steel formwork system previously laid out on the auditorium roof was then lifted and hung from the HSS truss system. The four PT concrete transfer level trusses were then constructed in a piece-wise manner, each piece adding to the strength and stiffness of the

PROJECT CREDITS

Owner: Vanderbilt University Medical Center
Architect: Donald Blair & Partners Architects, LLP, and Davis Brody Bond, LLP
Structural Engineer: Carpenter Wright Engineers, PLLC
Construction Manager: Turner Universal
Concrete Subcontractor: Charter Construction
Post-Tensioning Supplier: VStructural, LLC (VSL)
Post-Tensioning Specialty Engineer: Janssen and Spaans Engineering

PT hybrid steel and concrete truss, until the complete concrete trusses were in place and capable of supporting concrete floors at Levels 5 and 7. The HSS trusses were provided with the required camber (from both fabrication and subsequent post-tensioning) for the concrete trusses prior to the addition of the building structure above.

Because the structural steel formwork platform system and the detailing of the concrete trusses were developed simultaneously, the clearances required for the embedded HSS members were incorporated into the final member sizes and reinforcement detailing, as shown in Fig. 8. Integrated shop drawings detailing exact locations of all reinforcement, PT ducts, and anchorage assemblies, as well as the HSS formwork skeleton, were developed and thoroughly reviewed to ensure that the truss assembly would fit in the field.

To provide a safety check of the calculated truss member forces and to confirm that each subsequent tendon stressing stage could proceed, a minimum of two strain gauges were cast into the PT elements of each concrete truss. Readings from these gauges were taken after completion of each floor and before and after each PT tendon stressing stage. Construction was continued only after field measurements were reconciled with previously calculated values.

AIMING HIGH

The dedication ceremony for the opening of Medical Research Building IV was held in the Langford Auditorium on June 17, 2008. Vanderbilt University hopes that the expansion, of which this building is a key component, will help the Medical Center reach its goal of being ranked in the top 10 for funding from the National Institutes of Health by 2010.

References

1. *International Building Code 2000*, International Code Council, Falls Church, VA, Mar. 2000, 756 pp.

2. ACI Committee 318, "Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)," American Concrete Institute, Farmington Hills, MI, 1999, 391 pp.

Selected for reader interest by the editors.

ACI member **Otto J. Schwarz** is a Structural Engineer and Project Manager with Carpenter Wright Engineers, Nashville, TN. He is a licensed professional engineer with over 10 years of experience in the design of post-tensioned and conventionally reinforced concrete structures. He is a member of the Post-Tensioning Institute and a current board member of the Middle Region Tennessee Structural Engineers Association.