Design Considerations in Cable-Stayed Roof Structures

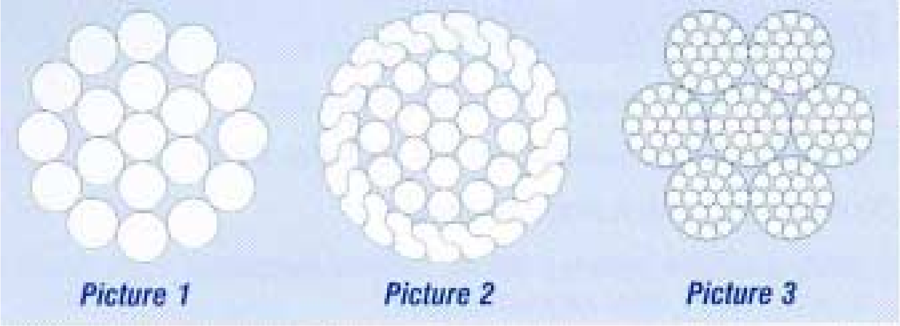
by Lawrence A. Kloiber, P.E., David E. Eckmann, AIA, S.E., P.E., Thomas R. Meyer, S.E., and Stephanie J. Hautzinger, AIA, S.E.

Starting with an overview of cable-supported structural technology, the authors elaborate on the engineering behind the University of Chicago's new athletic facility.

Cable-supported roof structures have inspired people for many years. However, cables systems are still a relatively new form of roof construction. Prior to the 1950s, steel cables were used primarily for long-span bridge structures, not buildings. Advancements in the understanding and analysis of cable-roof structures culminated in buildings like the Olympic Roof project designed for the 1972 Olympics in Munich, Germany. Today, cables structures are recognized as innovative structural solutions that create dramatic forms while efficiently enclosing large-volume spaces and providing transparency and natural light.

Cable-supported structures generally can be sorted into two categories: Cable-suspended structures, where draped cables are the main supporting elements of the structure, and their curvature is a major factor in the load-carrying capacity of the system; and cable-stayed structures, where cables stabilize vertical or sloped compression members (usually called masts or pylons) and serve as tension-only members.

The term "cable" generically is used to describe a flexible tension member consisting of one or more groups of wires, strands or ropes. Cables usually have yield strengths of approximately 240 ksi to 270 ksi. Cables are also inherently redundant members: Since they comprise dozens (or even hundreds) of wires, the failure of a single wire is not significantly detrimental to the load-carrying capaci-



Picture 1 is a "strand"—an assembly of wires wound around a central core. Picture 2 is a Z-lock cable, composed of Z-shaped cold-drawn or cold-rolled wires at the perimeter of the strand which lock together to "seal" the strand. Picture 3 is a wire "rope"—an assembly of multiple strands. Image from Pfeifer Catalog, 2000.

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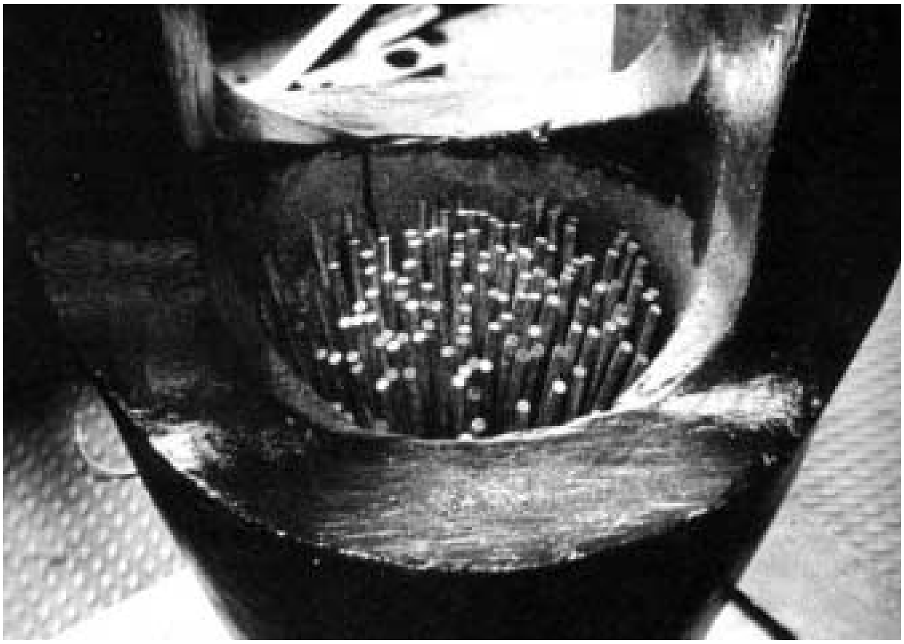


Broomed cable at cable and socket joint. Image from page 11, Harris James, Pui-K Li, *Masted Structures in Architecture*, Butterworth Architecture, 1996.

ties of the entire cable. A "wire" is a continuous length of steel that typically has a circular cross section, and is cold-drawn from a small-diameter steel rod. A "strand" is an assembly of wires formed helically around a central wire in one or more symmetrical layers. A wire "rope" is made from multiple wire strands that are twisted about a central core. Wire ropes frequently are used in cable-suspended structures because ropes are more flexible than strands.

Not all strands have wires with a circular cross section. Some cable manufacturers make a cable cross section called a full-lock or Z-lock cable, which consists of Z-shaped cold-drawn or cold-rolled wires at the perimeter of the strand. Full-lock cables are designed to reduce water infiltration in the cable assembly, reducing the potential for wire corrosion. Some cable manufacturers produce a proprietary full-lock cable with a zinc-rich powder that fills the small inner voids between wires, providing additional protection and prolonging the life of the cable. Since cable technology is relatively new, the life expectancy for cables in exterior building applications is not yet well defined. Many factors influence the life expectancy of cables, including climatic conditions, cable material properties, coating systems such as galvanizing and stainless steel, and use of cable sheathing and high-performance paints. However, by selecting an appropriate cable assembly and adhering to regularly scheduled inspections, many cables potentially can last as long as the structure they support.

Elasticity

Cables are elastic, yet they exhibit nonlinear behavior when loaded. The degree of nonlinearity varies with the cable structure as well as how the cables are loaded. The nonlinear effects of cables are generally less dramatic in cable-stayed structures than in cable-suspended structures. Two types of cable nonlinearities exist, geometric nonlinearities and material nonlinearities. A draped cable supported at two points in a horizontal plane will follow the catenary curve of the applied load and will undergo large geometric deformations, particularly when the load is concentrated or unsymmetrical. Geometric nonlinearities thus occur in loaded cables regardless of whether or not the cable material is elastic. Note that significant elongation of the cables and deformation of the supported structure must be considered in the design of cable-stayed roofs. A nonlinear analysis should be performed if the magnitude of cable displacements is such that the equilibrium equations for the structure should be based on the geometry of the displaced structure.

When cables initially are manufactured, they are not truly elastic, and often they are pre-stretched. If they are not pre-stretched, they will stretch inelastically as the cable is tensioned and individual wires settle into their final positions. Prestretching cables to a high percentage of their minimum breaking strength allows the wires to find their final position, with well-defined elastic characteristics. As helically wound cables are stretched, they will try to twist. To assure that cables are installed properly, that the cable length has not changed, and that they have not been twisted from their initial pre-stretched orientation, the designer should specify that cables be shipped and installed with a removable longitudinal stripe that clearly defines the proper cable-installation orientation.

Cables should be shipped on reels with sufficient diameter to prevent bending and loss of pre-stretching effects. Cables also should be protected at the site and handled to prevent kinking or other damage to the cables. They should be brought into place and handled by their termination fittings. There are several types of cable termination fittings available that "grip" the cable and allow it to be attached to the primary structure. Clamped, swaged, or socketed are typical classifications for fittings.

For example, a cable is inserted into the swaged fitting, which is placed in a die block of a hydraulic press. The softer steel of the fitting is hydraulically pressed such that the fitting's steel flows plastically around the harder steel wires of the cable. A swaged fitting is designed to develop the full strength of the cable, and is used with smaller-diameter cables (less than approximately 1½" diameter).

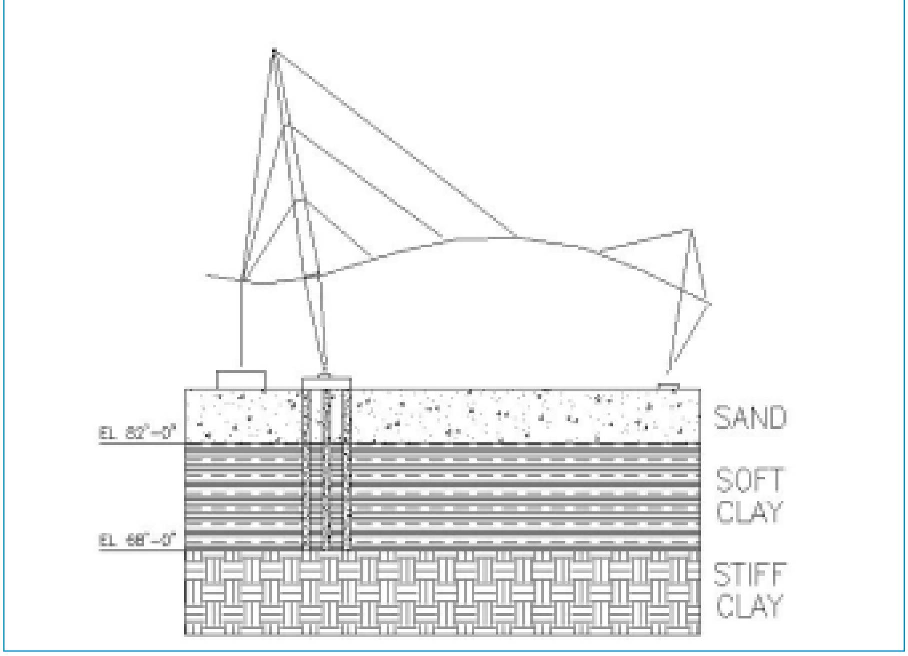
Sockets typically are used for large-diameter cables. Sockets are cast or forged steel shapes that are fully tested and provided by the manufacturer with the cable. The end of the cable is pushed into the socket, which has a wedge-shaped void to receive the cable end. Once the cable is in the socket, the cable wires are spread and separated within the wedge-shaped socket void, so that the cable end looks like a stiff broom. The socket void containing separated wires is then filled with molten zinc or resin. When the cable is tensioned, the cooled wedge bears against the inside surface of the socket, transferring the cable load to the socket. Socket connections are available in two profiles; open sockets, which have an opening to receive a single connection plate located on the structure, or closed sockets, which have a single end connection that is knifed between two connection plates on the structure. The cable fittings are designed to be stronger than the cables themselves. The termination fittings typically are designed to develop an ultimate strength of at least 110 percent of the cable strength.

References

Several books provide guidelines and commentary for cable-stayed structural design. However, governing building codes do not address specific design criteria for cable-supported structures. As such, the American Society of Civil Engineers (ASCE) developed useful standards through their publication ASCE 19-96, Structural Applications of Steel Cables for Buildings. The ASCE 19-96 publication offers recommendations for design drawings and specifications, design considerations, material properties, fittings, protective coatings, and fabrication and erection of cable structures.

ASCE 19-96 indicates that temperature effects on cables, vibrations, deflections, and erection analysis must be evaluated for cable structures. The ASCE standard also states the minimum breaking strength of cables shall always be at least twice the maximum cable design loads, including the envelope of loading combinations of cable self-weight, structure dead load, cable prestress forces, and live-load and environmental-load combinations. Cables also should maintain a minimum tensile force under all loading conditions to minimize visible cable sag and potential for induced cable vibrations. Maintaining minimum cable tensions is critical to achieving the stiffness necessary to stabilize the axial compressive masts and other structural components. Cable-supported structures are generally lighter structures and must be designed to account for the dynamic effects of individual cables and the overall structure.

The construction documents for cable-stayed roof structures typically provide information not shown on projects with more conventional structures: they define specific coordinates and parameters of the cable structure, including diameter and required cross-sectional area of the cables, which can vary depending on size and shape of wires used in the cables. The documents and specifications should indicate any requirements for wire coatings, unique material properties, and specific testing procedures. They also should identify acceptable tolerances for the final geometry of key coordinates of the erected structure, as well as ranges of acceptable final cable tensions at a defined ambient temperature.



Triple-tube jet grouting beneath the mast foundations will minimize long-term settlements.

The design documents also should provide parameters for the erection sequence. While it is the erection engineer's responsibility to establish the erection sequence, the design engineer is most familiar with the final structure, and can provide valuable information regarding issues of load flow and structural stability. The design engineer should be aware that the erection sequence might require cable forces not analyzed originally for the completed structure. When cables initially are tensioned, frequently they are not tensioned to defined design tensions. Unless all cables are tensioned simultaneously, the magnitude of cable tensions will change as subsequent cables are tensioned. As a result, some cables initially are over-tensioned and others under-tensioned to achieve the correct final tensions.

Case Study: Ratner Center

The University of Chicago is in the midst of its largest capital development program in its history, and is investing more than $500 million in new facilities and renovations. The new Gerald Ratner Athletics Center is a $51-million state-of-the-art athletics facility with 150,000 sq. ft of fitness and sports area. The project includes a competition gymnasium and an Olympic-sized natatorium.

Cesar Pelli & Associates teamed with OWP/P to design the facility. OWP/P served as the architect of record for the project, and OWP/P Structures, a division of OWP/P, was structural designer and structural engineer of record for the project.

The Ratner Center is the first asymmetrically supported cable-stayed building with multiple levels of splaying cables in Chicago, and possibly the world. The structural solution allows large-volume spaces (more than 20,000 sq. ft) to be enclosed with structural steel members that are only 33″ deep.

Structural Overview

The structural system for the gymnasium and natatorium space is a masted cable-stayed roof system of composite masts that are sloped, tapered, and stabilized by 15 cables (nine fore-stay cables and six back-stay cables), which in turn support S-shaped roof girders.

The structural system allows column-free spaces of 160′ by 125′ in the gymnasium, and 130′ by 200′ in the natatorium. Both spaces have a similar system of primary masts spaced at 75′ on center, which are opposed at the other end of the structure by smaller masts located at 25′ on center. Each primary mast supports three curved roof girders, with the mast located on axis with the center girder. The cables splay outward from the mast, and support roof girders located 25′ to either side of the central girder. The cables are connected to the primary mast at three distinct elevations: top of the mast, approximately 25′ below the mast top, and at approximately 50′ below the mast top. Each level has three forestay cables that reach to one of the three supported roof girders. Each level also has two backstay cables, each reaching to a tie-

The Gerald Ratner Athletics Center at the University of Chicago consists of two asymmetrically supported cable-stayed buildings with multiple levels of splaying cables.

down connection located 25′ on either side of the mast. The backstay cables transfer load to steel tension columns, which in turn anchor to massive concrete foundations resisting the overturning forces from the weight of the roof. Smaller secondary masts opposite the primary masts are located at each girder line. A single cable from the top of this mast supports a portion of the roof girder. Two back cables in a bow configuration help resist the flattening tendency of the arched roof. The result is a series of roof girders supported at four points by cables, effectively reducing the girder span and allowing the girders to be only 33″-deep wide-flange sections.

The asymmetry of the gymnasium and natatorium structure is a result of site constraints and architectural design preferences. The asymmetry results in an unbalanced horizontal thrust delivered to a truss located in the roof plane behind the primary masts through axial load in the curved roof girders. The horizontal truss transfers load to the building's vertical braced frames, essentially making the lateral system a component of the gravity system.

With each of the primary masts supporting three separate roof girders, the load supported by each mast must be balanced. Thus, the girders located at the exterior walls on each side of the structure must be supported by the mast and cable system, and not the perimeter columns located below the end girders. The columns at the exterior sidewalls of the spaces offer no vertical support to the roof structure, and are connected to the roof only with vertically slotted connections. The sidewall columns primarily are vertical beams, which resist wind loads from the 50′-tall exterior walls. However, the perimeter columns are also components of the lateral system, and so the vertically slotted connections can transfer horizontal loads from the roof diaphragm. The roof is essentially a curved floating plane suspended only by the cable system. It moves approximately 3.5″ up and down at mid-span under the envelope of loading conditions across the 160′ span of the gymnasium, and 3″ up and down across 130′ span of the natatorium.

The masts are inclined at 10 degrees from vertical for aesthetic reasons and to maximize effectiveness in the asymmetrical structure. To maintain the mast as a predominantly axially-loaded member, the base of the mast is modeled and detailed literally as a "pinned" base in the direction of the span, such that the slight rotation of the mast under loading conditions does not induce moments at the mast's base. Additionally, the W33 that is in line with the mast and carries axial horizontal thrust to the roof truss located behind the masts is detailed as a collar that passes load around the mast, minimizing bending in the mast. Each mast in the gymnasium transfers more than 1700 kips of vertical load from the roof through the cables to the foundation.

Since the large-volume gymnasium and natatorium spaces could not be interrupted with interior columns or a tie-down system, a thin layer of concrete topping was added to the long-span roof deck to provide sufficient dead load to offset uplift forces from gusting winds. The roof deck is a 7½″-deep, long-span deck with 2½″ of lightweight concrete topping. It spans 25′ between the W33 roof girders. The deck acts as a lateral diaphragm in the direction perpendicular to the span of the W33s, transferring load to high-strength diagonal bracing rods at each end of the gymnasium and natatorium spaces.

Composite Masts

The 10-story-tall masts are composed of three 18″-diameter steel hollow structural sections (HSS) arranged in a triangular, tapering form, and tied together with 12.75″ HSS at elevations of cable attachment and at the point where they pass through the roof (roughly 30′ above the base of the mast). At their widest, the composite masts of the gymnasium are 7′-6″ across, and 5′-9″ across at the natatorium. The secondary masts are 54′-tall, single HSS of the same diameter. Each of the 18″-diameter sections is filled with 10,000 psi high-strength concrete acting compositely with the steel to resist the compressive loads delivered from the cable-stayed roof.

The contractor placed the concrete in a single concrete cast through "ports" located at the roof level. A high-slump cast-in-place concrete was allowed to free-fall 30′ to the base of the mast, then was pumped up the remaining height of the in-place masts, reducing the possibilities of air pockets and voids that could have formed around the internal stiffeners located points of cable attachment. This was a constructability challenge and required specifying an appropriate mix design. To ensure the integrity of the concrete during the winter's freezing temperatures, particular attention was paid to appropriate cold-weather concreting procedures.

The hollow steel masts were fabricated and shipped in one piece to the project site, where they were lifted into place by a high-capacity crane. In early design, the engineering team researched availability of large-diameter HSS and found that 18″-diameter, half-inch wall section of ASTM A500B material was available from select producers, and proceeded with design. However, at the time of material purchasing, insufficient quantities of this section were available, and a substitution was made to use the more readily available API 5L × 42 line pipe. Additionally, cable connections to the mast were required to be designed for the cable's minimum breaking strength, not only the cable design load. The cable breaking strength is at least twice the maximum allowable cable design load, and in some cases a heavier-wall (.812″), higher-strength (65 ksi) section was spliced into the mast at cable connection points to reduce local stresses on the HSS walls. In other cases, a system of internal stiffeners was designed to adequately transfer the cable loads to the composite mast.

Both the natatorium (shown) and the gymnasium feature cable-supported roof structures.

Cable Design Considerations

Under all loading conditions, it was critical that the cables maintained a minimum amount of tension to mitigate noticeable sag, to minimize detrimental vibrations, and to provide stability to the masts. Cables were pre-stressed to ensure they always would be in tension. Cable tensions under numerous loading conditions, including snow loading, snow drifting, and wind uplift and downward forces were analyzed. Also important was analysis of thermal impacts throughout the full range of climatic temperature variations on the structure, ice loads, and predicted long-term settlement of the mast bases. RISA 3-D and ROBOT structural analysis software were used to develop three-dimensional structural models to investigate load combinations and to account for any non-linearity of the structure. The goal was to achieve tensions that fell within the desired envelope to meet minimum serviceability tensions, not exceeding maximum allowable cable tensions under all loading conditions throughout the structure's life. Per the applicable cable standards, maximum cable tensions should not exceed 45% to 50% of the cable design strength. Galvanized cables were used, ranging in size from 36 mm to 66 mm in diameter, with minimum breaking strengths of 286 kips to 978 kips.

To minimize maintenance and increase longevity, full-locked cables were specified to reduce water infiltration and subsequent corrosion of the cable wires. The helically wound cables include two to three outer layers of interlocking Z-shaped wires, specifically designed to inhibit water infiltration, surrounding a circular wire core. Full-locked cables currently are not produced domestically, and all of the cables were imported from Germany.

Mast Stability Analysis

The composite masts are critical compression elements and were subject to complex stability considerations and analyses. The masts are not symmetrically braced about their vertical axis, but are braced at multiple levels by tension-only elements (cables) of varying stiffness. The spring braces of varying stiffness are provided by the cables tied to the foundations behind the mast, and the cables connected to the W33 girders, which also are spring supports. This presented stability and buckling issues for the masts and the W33 girders (which also resist axial loads) not directly addressed by current design codes. The designers checked references and consulted with stability experts to establish an appropriate analysis procedure to calculate axial capacities and critical buckling loads of these members. Each composite tied-column mast has varying axial load along its length and between each of three distinct legs, and biaxial moments. Structural analysis software aided in determining spring stiffness at each cable connection level, which was reduced by the calculated spring stiffness of the W33 girders. This informa-

Above, left and right: The 10-story-tall masts are composed of three 18"-diameter steel hollow structural sections (HSS) arranged in a triangular, tapering form, and tied together with 12.75" HSS

tion helped determine brace forces. It also was compared to required brace forces to determine k-values for each mast segment. Ultimately, axial capacities and moment capacities of each segment were calculated to evaluate actual maximum loads to ensure satisfactory design of each mast.

Redundancy

Since the City of Chicago Department of Buildings reviewed the project for a building permit shortly after September 11, 2001, the City required that structural engineers closely review the redundancy of the structure. Since clear redundancy criteria for this type of tensile structure had no precedent, the City established criteria requiring the investigation of instantaneous cable failure and associated effects. The structural design, including stability of the mast with a missing cable, was evaluated and deemed satisfactory.

Reverse Curved Roof Girders

The architectural design called for the natatorium and gymnasium to be enclosed by curved S-shaped roofs. Each of the W33×169 roof members, 160′ in the gymnasium, 125′ in the natatorium, is cold-bent about its strong axis with reverse curves to multiple radii. Segments of W33 up to 100′ long were fed through a series of rollers to achieve the specified radius in just minutes.

Wind Tunnel Testing and Analysis

The reverse curvature of the gymnasium and natatorium roofs present challenges when applying code provisions for wind and drifting snow. Current codes and standards do not specifically address wind-loading and drift criteria for the unusually shaped roof. Wind-tunnel and water-flume tests were performed to provide design parameters for the cable-stayed structure. An accurate measure of the drift and wind-loading patterns allowed designers to further optimize the building design. It also identified heavily loaded areas that are not intuitive or addressed by the governing code. For example, the water-flume test identified a snowdrift configuration that could form on the downward-sloping portions of the roof near the eave, a critical loading condition for the long-span roof deck and the overall structure.

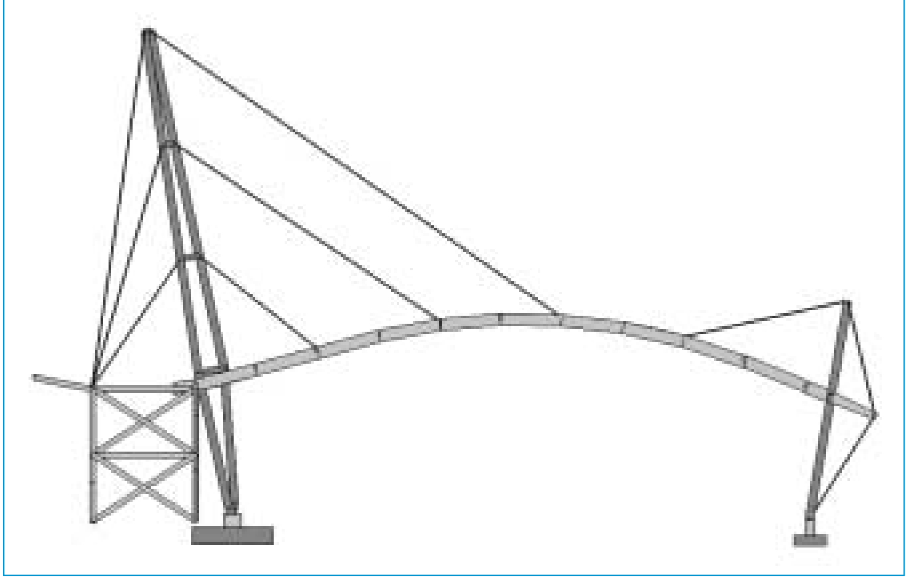
Further complicating the matter is the cable structure's sensitivity to unbalanced loading conditions as well as its dynamic response to lateral loads. The wind tunnel test results provide the means to ensure a clear and reliable understanding of the structure's response to unbalanced loads. A dynamic analysis of the structure verified that the building's fundamental frequency does not align with the frequencies of the individual cables, minimizing concerns about harmonics within the structure.

Mast and Tieback Foundations

The behavior of the cable-stayed structure is sensitive to settlement of the supporting foundation, particularly at the mast base. Excessive settlement of the mast foundation could reduce cable tensions below the envelope of acceptable tensions, and selecting the appropriate foundation system for the mast and tieback columns is critical. The soil profile at the site consists of a top layer of sand approximately 15′ deep. Below the sand is 15′ of soft clay underlain by stiff clay. The relatively high bearing strength of the sand allows for a reasonably proportioned conventional shallow foundation at the masts. However, due to the magnitude of the mast's sustained loads, the soft clay could experience long-term settlement of undesirable magnitude if not addressed in the design. Since the foundations for the other elements, like the secondary masts, resist substantially less sustained load than the masts, conventional shallow foundations were appropriate for the remainder of the structure. The challenge was to find a mast-foundation system compatible with the long-term anticipated settlement of the rest of the structure. Large differential settlements between the masts and the rest of the building could cause the cables to lose tension and experience visible sag.

Traditional deep-foundation systems were deemed not to be cost effective. The geotechnical engineer recommended a soil-improvement method called triple-tube jet grouting, which was implemented to limit the long-term settlement of the mast foundations to one-half inch. This technique uses a mixture of grout, water, and air injected under high pressure to create "columns" of a soil-grout mixture. These columns transfer the loads from the foundation directly into the stiff clay layer.

The asymmetry of the cable structure results in large uplift reactions at the base of the tieback columns. Deep foundations relying on friction or soil weight were explored as alternative solutions, but were determined not appropriate or not cost effective. Concerns with deep foundations included potentially adverse impacts of long-term upward settlements on the behavior of the cables, as well as highly tensioned elements in the ground that could possibly be disturbed in the future. In addition, several tieback



Gymnasium section.

Pinned-base detail at masts.

columns are closely spaced and areas of influence overlap.

These considerations directed the design team toward the solution: reinforced-concrete counterweights buried below grade that utilize their self-weight to resist the uplift forces from the cable-stayed tension elements. The system ensures a foundation with no upward displacement over time, and a system that will not be impacted by future removal of the surrounding soil. The long-term groundwater level limits the depth of these foundation elements to approximately 10′ below grade. If the concrete counterweights extend below the water table, the hydrostatic pressure on the bottom of the footing produces additional upward forces. As a result, the counterweights are large in plan relative to their depths. In some cases, the depth restriction forces two tie-back columns to anchor to the same counterweight foundation, with footings that reach sizes of 25′ × 50′ × 8′ deep.

Cable-Stayed Roof Erection Procedure

ASCE 19-96 *Structural Applications of Steel Cables and Buildings* suggests the contract documents show a recommended erection procedure for the proposed structure. During the design phase, the challenge was to get the final structure to the specified geometry and cable tensions, knowing that each stage of the erection sequence would impact the geometry and cable tensions. These discussions helped shape the procedure documented on the structural drawings.

The original concept on this project was to fabricate the structure to a specified geometry at a defined ambient temperature. Temporary shoring towers at each cable-attachment location were to be used to keep the structure in the required geometry prior to cable tensioning. Once the entire structure was erected and supported by the shoring towers, the cables were to be tensioned simultaneously, lifting the structure off the shores and resulting in the intended geometry and cable tensions. This approach has been successfully on other projects, including the cable-supported fabric roof at Stuttgart Stadium in Germany.

Once a fabricator and erector were awarded the project, the erector investigated alternate erection scenarios, particularly erecting the structure in stages, reducing the need for simultaneous jacking. The project specifications required an erection engineer, a structural engineer licensed to model the structure and the erection procedure. The erection engineer chose to erect the structure in stages and had to model the erection procedure using staged non-linear analysis techniques to ensure the final specified geometry and cable tensions were achieved. The erection engineer considered an erection procedure where all the cables were attached to the mast, and the roof girders would be hung directly from the mast-supported cables, eliminating the need for temporary shoring. However, that scenario induced unacceptable horizontal thrusts on the masts from the roof girders.

The contractor's favored solution was to erect the roof girders using a single temporary shoring tower near the midspan of each roof girder. Once erected, cables are connected to the mast and roof girders, and are tensioned in stages, starting at the lowest level of splayed cables. The same procedure was implemented at the adjacent masts before the crew moved up and began tensioning cables at the next-highest level of each mast. Cables were tensioned with two hydraulic jacks pressurized in parallel, attached to the two threaded rods on each socket. Pressure gauges on the jacks allowed the erector to covert pressures and determine the exact force in each cable.

Conclusion

Cable-stayed roof structures can create dramatic structures that enclose large-volume column-free spaces, and still provide opportunities for architectural design freedom. The University of Chicago Gerald Ratner Athletics Center is an innovative cable-stayed structural system. The structural solution reduces the depth of the roof members and project cost by using cables to suspend the structure. The roof system also serves as the building's finished materials, eliminating the need for costly ceilings and other cladding materials. The cable-stayed solution creates a thin structure that "floats" over the interior spaces, and allows natural light to penetrate the spaces through continuous clerestories located along the roofline. ★

*This paper has been edited for space considerations. To learn more about cable-stayed structures, and the design and construction of the University of Chicago's Gerald Ratner Athletics Center, read the complete text online at **www.modernsteel.com** or in the 2004 NASCC Proceedings.*