DESIGN OF CONCRETE TRACERY TRUSSES TO DEFINE ARCHITECTURAL SPACE

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ABSTRACT: Exposing the building's structure and using it to define space was frequently practiced in earlier eras, often giving rise to innovative structures that integrated with the architecture. Engineering is rarely used today, however, to achieve such results. This paper examines a recent project in which the integration of structure and space was an explicit goal, and discusses the design and construction of the exposed concrete "tracery" roof trusses used to achieve this end. There are significant structural, cost, and construction problems associated with this class of structures and these are discussed. Further, some of the key aspects of integrating structure and space, and of a design process that makes the design task practical are briefly presented.

BACKGROUND

In March of 1987 the Center for Environmental Structure, Berkeley, Calif., an architecture and engineering firm in which the writer is the chief engineer and vice president, was commissioned to design a 50-bed, 1,600 m^2 (16,000 sq ft) homeless shelter for the city of San Jose, Calif. The construction budget was moderately low for California at $1.1 million, or $690/m^2 ($69/sq ft). The owner requested a building that would be welcoming and noninstitutional, but at the same time would be durable and able to stand up to the wear and tear imposed by the patrons, many of whom are aggressive. As an overall design strategy, to address the demands of the client and cost constraints, the writer proposed to spend a higher percentage of the budget on the structure, (a strategy that could help ensure durability) and expose it throughout the building to achieve architectural goals (an approach that could make double use of the structure and help keep the project within the limited budget). Adoption of this philosophy resulted in a design aesthetic in which the structure, consisting principally of heavy timber, cast-in-place and precast reinforced concrete, and concrete block, was exposed throughout the complex, and was designed and detailed to give solidity and architectural form to the various spaces (Fig. 1). This approach was used in the main passages and arcades, dayrooms, entrance lobby, and reception but it found its most dramatic expression in the roof trusses of the dining hall (Fig. 2).

STRUCTURE AND SPACE

The integration of structure and space is a recurring theme in the great buildings of the past. In cases, such as Chartres Cathedral (France, 1194–1260), Hagia Sophia (Turkey, 532–537) and the Stave church at Borgund (Norway, 1150) the structure performs a dual role—that of defining and creating architectural space at the same time that it provides the structural framework. In these buildings, the architecture and the engineering is conceived as being one task, probably not so much because of an attempt to "integrate" them, but because they had not yet been divided. As one author wrote,

> . . . I have chosen to start this narrative in the late eighteenth century . . . before then, the principal building materials were stone and wood, materials in which it is difficult to separate structural from architectural design," (Billington 1983).

The significant feature in these examples is that the precise geometrical organization of the structure, both at the global level and at the detail level, is attuned to define space while concurrently performing its structural role.

Tracery

The initial inspiration for the homeless shelter's concrete trusses came from two sources: the great traceried roof trusses at Westminster Hall (London, 1395) designed and built by Hugh Herland and recorded more extensively by Baines (1914), and the decorative tracery in the Chapel of the Chimes (Oakland, Calif., 1926–30) designed by Julia Morgan. In the Westminster Hall truss (Fig. 3), the proportions, detailing, and placement of the various structural elements

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FIG. 1. Homeless Shelter, San Jose, Calif., 1989; Passage and Court with Exposed Timber Framing, Precast Concrete Columns, and Concrete Block Walls: Design by R. Gary Black and Chris Alexander of the C.E.S. (Photo © Mark Darley/Esto)

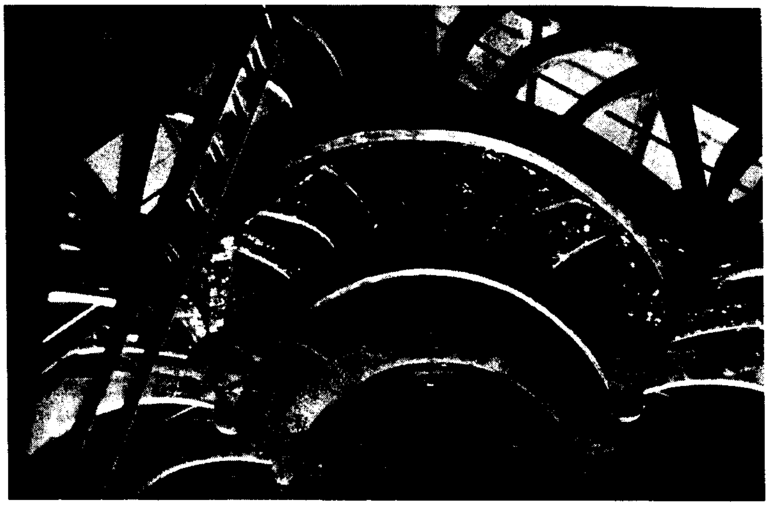

FIG. 2. Homeless Shelter, San Jose, Calif., 1989; Reinforced Concrete Roof Trusses just after Removal of Forms: Design by R. Gary Black and Chris Alexander of the C.E.S.

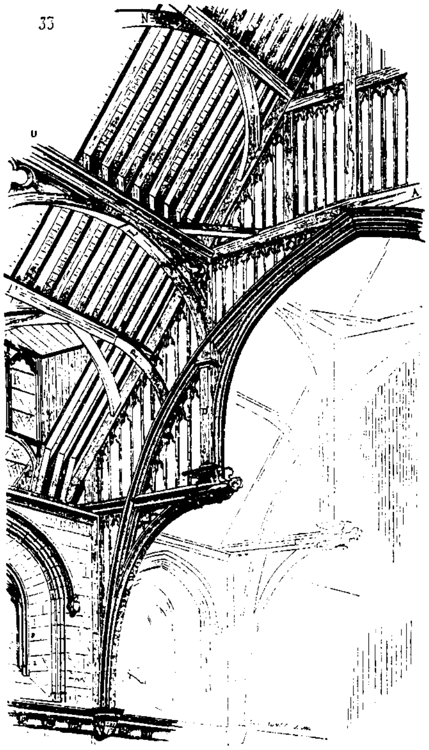

FIG. 3. Westminster Hall, London, 1395; Drawing by E. E. Viollet-le-Duc: Design by Hugh Herland ("Charpente" 1859)

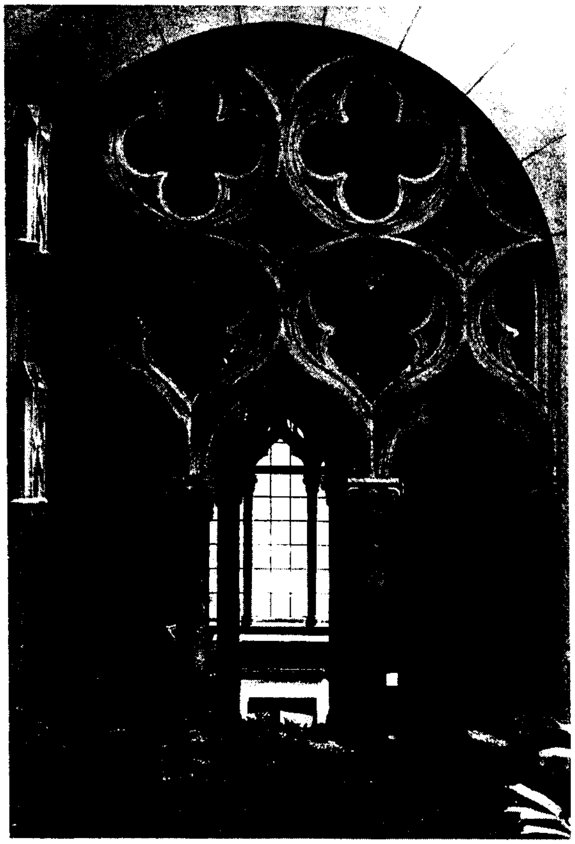

FIG. 4. Chapel of the Chimes, Oakland, Calif., 1930; A Visit to This Building in 1982 Prompted the Writer's First Investigations into Using Reinforced Concrete Tracery in Structural Applications: Design by Julia Morgan

(hammer posts, hammer beams, great arch rib, wall posts and corbel, bracing struts, wind braces, and tracery) do much more than just carry gravity and wind loads, they simultaneously give form to the room through their interlock with the space they define. Modern engineering analysis of this 600-year-old structure undertaken by various researchers, Heyman (1967), Mainstone (1967), Courtenay and Mark (1987), Black and Morris (unpublished, results presented at the 1993 meeting of the Society of Architectural Historians, 1993) indicates that very little of the structure, if any at all, is superfluous. The design, a very particular form of integral ornament that is largely responsible for defining the formal expression of the architecture, is essentially pure structure creating architectural space.

On a smaller scale, in the Chapel of the Chimes (Fig. 4) sand-cement mortar tracery was used to define and articulate the space in passages and around openings. In this case the tracery was nonstructural, however, during a visit to the building in 1982 I first became aware of the structural and spatial potential for using thin elements of expose reinforced concrete as structural tracery and began then to devise various applications for it. It is worth mention that nearly a hundred years earlier the architect Anatole De Baudot had employed a similar system devised by the engineer Paul Cottacin and in collaboration with him designed and built Saint-Jean-de-Montmarte (Paris 1894–1904) a church constructed of an extremely thin [7.65 cm (3 in.)] net of reinforced cement members, bonded into the reinforced brick side walls and facings (Fig. 5), (Frampton and Futagawa 1983).

STRUCTURAL BEHAVIOR

Two structural issues that are not normally considered in truss design are relevant to the design of concrete trusses. The first involves a problem of moment connections at the joints, and the second pertains to out-of-plane inertia forces associated with the relatively large self weight. Additionally, problems related to long-term deflections and redistribution of forces caused by creep could be problematic in some structures and should be considered and accounted for in the design. Each of these problems is presented here within the context of the concrete trusses for the homeless shelter; they are also of such a general nature that any use of concrete tracery constructions demands their resolution.

Moment versus Pinned Connections

The basic assumption behind the design of modern trusses, either that all the members behave as two-force elements with pin joints or that they behave as Vierendeels, proved inadequate in the design of these trusses. Even when a conventional truss is not fabricated with perfect pins, (i.e. some of the joints can transmit moment via gussets or continuity of an element through a joint) the physical behavior still will approximate that of the analytical model within a few percent by virtue of the geometry that necessarily results from the basic design assumption. In contrast, and at the other end of the spectrum, Vierendeels, which assume rigid joints and replace the pin-jointed diagonal members with moment connections for shear transfer, assume orthogonal geometries that do not include diagonal elements. Many of the early hammer-beam trusses, such as those at Westminster, however, rely on a structural behavior that lies somewhere between these two extremes and cannot be designed or analyzed on the basis of either model alone.

In general, trusses are defined as "combinations of purely tensile and compressive elements capable of spanning . . . and a plane truss consists of straight bars, usually of constant cross section" (Salvadori and Levy 1981). This definition necessarily leads to specific geometries that result in an internal force field that is consistent with the definition. When a truss structure is

FIG. 5. Saint-Jean-de-Montmarte, Paris, 1904; Interior of Church Showing Use of Ultrathin Reinforced Cement Structure: Design by Anatole De Baudot and Paul Cottacin (Frampton and Futagawa 1983)

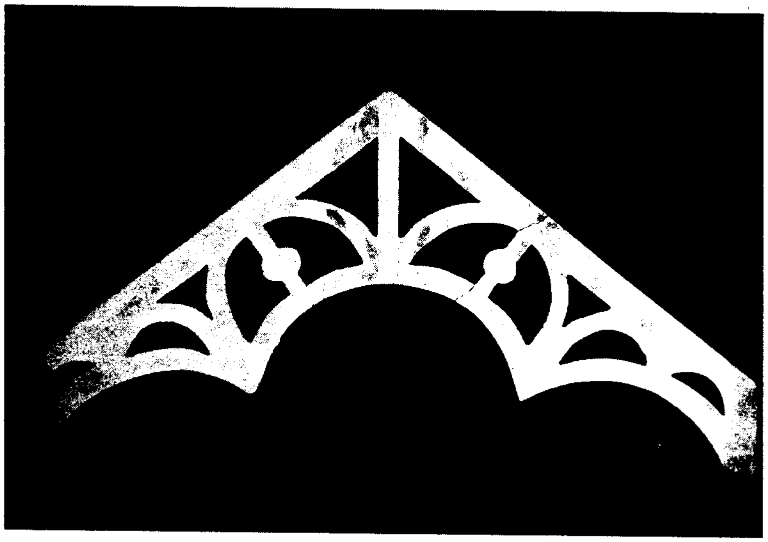

FIG. 6. Early Truss Design That Was Abandoned because of Excessive In-Plane Moments

designed that breaks with these geometries, contains curved members, or members of varying cross section, it will no longer exhibit the strict behavior represented by the preceding definition. If the bottom cord of a roof truss is fabricated from a series of steps and offsets, then the structure will most often have to rely on the presence of both pin and moment connections to ensure structural stability. In this regard, the trusses of the homeless shelter have more in common with early hammer-beam trusses than with the conventional definition.

In conventional trusses made of wood or steel, the designer can create either moment or pin connections through simple detailing of the joint. This permits control over the flow of forces in the system, eliminating unwanted moments by insertion of a pin. A concrete truss, on the other hand, does not easily permit this since pure pins are virtually impossible to create in cast-in-place concrete.

The writer initially designed and analyzed the truss using a completely different arrangement of web members assuming moment connections at every joint (Fig. 6). This strategy proved ineffective because whenever a member was up-sized to handle the moments, its increased stiffness caused a global force redistribution that required further up-sizing. To address this problem different arrangements of web members having tapered ends to reduce their moment of inertia was tried as a technique to allow some joints to behave in a pin-like manner. This proved to be an effective approach in balancing forces and member sizes because it allowed for selective insertion of pin-like joints while leaving other moment connections unimpaired. The final layout and geometry of the web members, which are arranged to behave principally in tension or compression, are tapered toward their ends and reinforced using a single 32-mm (#10) rebar so that upon local cracking secondary moments (resulting from joint rotations) do not induce unwanted moments and shears in the structure.

Out-of-Plane Forces

In trusses, made from wood or steel, self weight of the structure is a relatively small fraction of the total load, and the out-of-plane stresses induced by the self weight during seismic excitation is rarely problematic. Concrete trusses, on the other hand, are significantly heavier, having a relative weight-to-strength ratio about four times that of solid-sawn wood and six times that of mild steel. In high seismic zones, therefore, it is necessary to consider and resolve the out-of-plane inertia forces. For the design of the homeless shelter trusses, an equivalent static load factor was determined from a simplified dynamic analysis. The dynamic analysis assumed a simply supported beam, with equivalent span and stiffness to the truss, loaded in the out-of-plane direction with three lumped masses totaling the self weight of the truss. This analysis indicated maximum out-of-plane forces equal to about 40% gravity. In the computer model 40% gravity loads were therefore applied statically as a distributed force to each frame member in the out-of-plane direction.

Under the prescribed loading and support system the bottom cord, which is thicker than the web members, behaves like a beam spanning the distance between supports. As the bottom cord deflects out-of-plane it also rotates, inducing moderate bending in the web members, which causes rotation of and torsion in the top cord. The resulting torsion and bending stresses, although of an apparent secondary nature, needed to be accounted for in the reinforcement detailing because in a further analysis in which the out-of-plane bending action of the web members was released the bending stresses in the bottom cord resulted in unacceptable cracking.

To mitigate the out-of-plane deflection and attendant cracking, two measures were used. First, the span was reduced by providing lateral support at the junction between the central arch and the two side arches. This was achieved with two diagonally placed 22-mm (7/8-in.) steel rods running from the bond beam at the head of the wall. The attachment at the wall was made by drilling and epoxying, and the connection at the truss was achieved by threading and thru-bolting the end of the steel rod (Fig. 7). In this way, a slight tensile prestress could be induced into the rod to limit lateral deflection causes by loose connections. The second measure was to further reduce the weight by reducing member sizes and specifying a lightweight concrete. Concrete specifications called for 34,500 kPa (5,000 psi) compressive strength at a density of 1,700 kg/m^3 (110 pcf), which is about the maximum strength-to-weight ratio that can be obtained using pneumatically placed concrete.

Creep

Creep was considered as a potential problem, but a detailed analysis was not undertaken. In discussion with Professor Vitellmo Bertero and Edward Wilson of the Department of Civil Engineering at the University of California, Berkeley, however, a concern was raised about possible problems with creep resulting from the complex geometry of the structure. They were concerned that as a result of creep, a redistribution of forces might cause the bottom cord to behave as an arch, buttressed by the wall and pilasters of the building. In such a case, there would be potential for out-of-plane buckling of the arch. To address this concern, the truss was designed to be supported on rollers, a condition that was achieved in practice by slotting steel

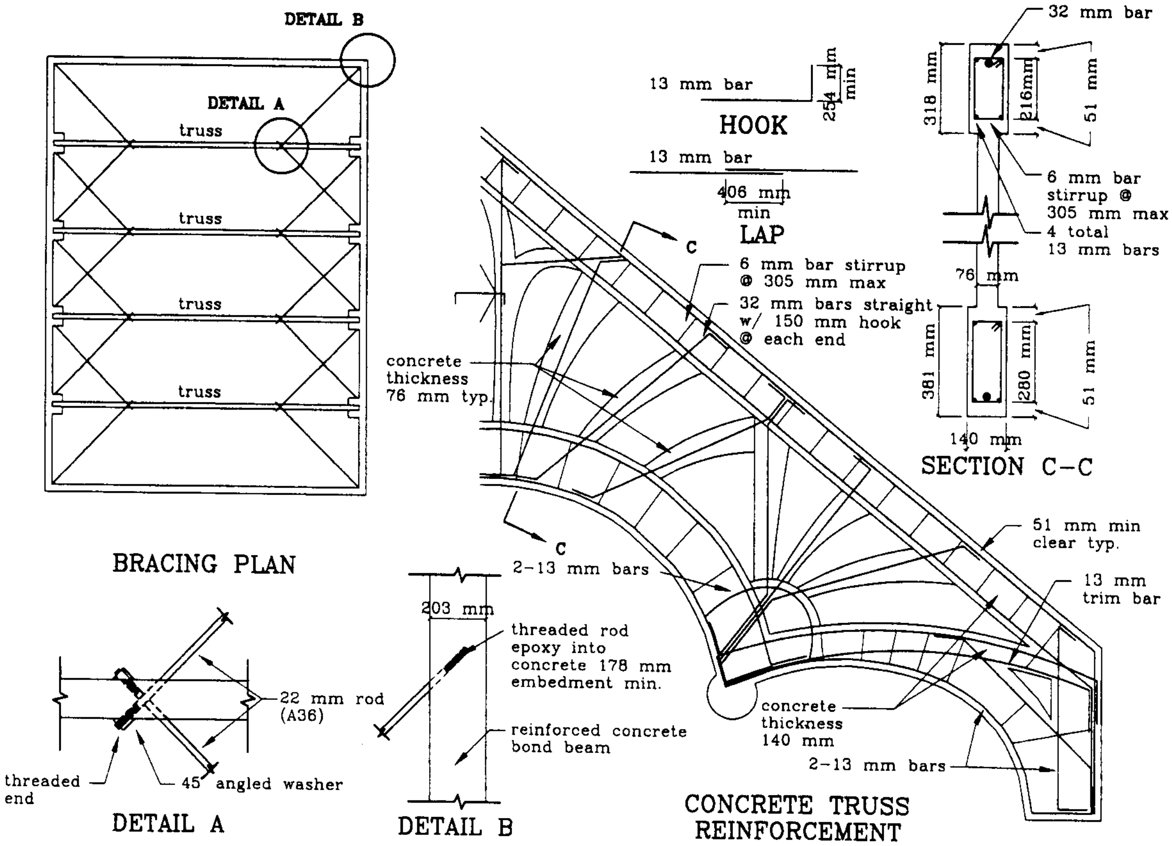

FIG. 7. Principal Reinforcement for Truss and Details of Bracing Scheme Used to Reduce Out-of-Plane Bending Caused by Seismic Excitation

bearing plates at each end of the truss at the points of bolt penetrations, supporting the entire structure on teflon pads, and leaving a space between the end of the truss and the wall. This allowed the ends to slide, thus maintaining tension in the bottom cord at all times.

CONSTRUCTION ALTERNATIVES AND COSTS

Two related construction issues were finding construction procedures that would ensure structural integrity of the in-plane trusses and finding procedures that would keep the costs to a minimum. In this design, the most economical forms of construction were impossible because the procedures could have caused structural damage to the trusses.

Construction Methods

Of the various construction alternatives considered, three were ultimately investigated in depth: pouring flat on a casting bed and erecting in place, pouring in place, and guniting in place. Structural analysis, performed to study the out-of-plane stresses in the truss during lifting showed that a problem arises with the method of pouring on a casting bed due to the combination of large self weight and relative weakness in the out-of-plane direction. Assuming the trusses are formed and poured on plywood forms resting on the existing slab, at the time the concrete is poured, the structure will be fully supported on an essentially rigid support. Consequently, as the truss is lifted off of the slab it will deflect in proportion to the number of support points (between the lifting harness and the truss) and the overall stiffness of the harness itself. In the computer studies, the lifting harness and points of attachment were included as part of the model.

The design criteria for lifting assumed full dead load and required that no portion of the concrete be allowed to experience an extreme fiber tension stress in excess of 2,800 kPa (400 psi) to diminish the amount of tensile cracking. Under these assumptions, the lifting harness design called for minimum depth 305 mm (12 in.) steel sections and required 20 support points on the truss. The rather large steel sections were required for stiffness, not strength. It should be noted that these design criteria assumed ideal conditions, in which the truss would not be loaded in excess of 100% of its dead load—a somewhat optimistic assumption given the possibility of sudden release from the forms and the general dynamic nature of lifting operations. As a result of these studies, this method was abandoned because of expense and the concern that the in-place structure might contain numerous hair-line cracks that could reduce its performance in an earthquake.

Since these studies, the author has had the opportunity to design other tracery trusses (in one case, twelve units forming segments of a twelve-sided dome). Based on preliminary design work, it appears the lifting problem can be solved by forming and pouring the structures on a "soft" casting bed. In this method, the forms would be supported at specific points which would correspond exactly to the pick-up points on the structure. Depending on the specific design of the harness and the exact methods of lifting, little or no out-of-place stress would be induced in the structure during erection.

The second construction alternative involved forming and pouring the structure in place, an approach that avoids the problem of out-of-plane bending during lifting but which is made impossible by the thinness of the 76-mm (3-in.) webs, the density of reinforcing steel in the upper and lower cord members, and the slump required of high-strength concrete.

The third approach, and the one finally adopted, involved building forms on the ground that were open on one side. The reinforcing steel was subsequently placed in position and attached to the forms and the entire assembly hoisted into position [Fig. 8(a)]. Once installed and braced, concrete was shot into the forms using pneumatic placement techniques [Fig. 8(b)]. Of the two commercial versions available, shotcrete (wet mix) and gunite (dry mix), the latter was selected because the quality control with respect to compressive strength is greater and because less shrinkage cracking occurs with this method.

Costs

Each of the construction methods just described were evaluated on the basis of cost. The first method, casting on the slab and lifting into place was economical only if the number of pick-up points could be substantially reduced. Forming and pouring in-place costed out as less expensive than the gunite version, but the gunite method was adopted because it was the only one that could ensure structural integrity of the completed trusses. Further measures were taken within this approach to reduce costs. One-sided forms with reinforcing were built on the ground because it was less expensive than forming in place. In addition, two scaffolds were built (instead of four) one between each pair of trusses so that two trusses could be shot from one location. The downside is that an uneven finish results on either side of the trusses, as the exposed side has a different texture from the one against the form.

When completed, the four trusses cost $24,000 of which $10,000 was spent on the guniting operation (approximately $1,000 per cubic yard for the concrete), and $14,000 was spent for forming, placement of steel, scaffolding, and stripping. By comparison with another totally different approach, the bid from a glue-lam manufacturer to make the bottom cords (as three separate arcs) was $32,000. When the remaining work of connections, fabrication of the webs, assembly and installation was added in, the total cost of the trusses, in wood, was estimated at $47,000 or twice the cost of the concrete solution. Prefabricated wood trusses with a flat drywall ceiling would have cost approximately $5,000.

Although the concrete trusses as designed represent the most cost-effective approach to the

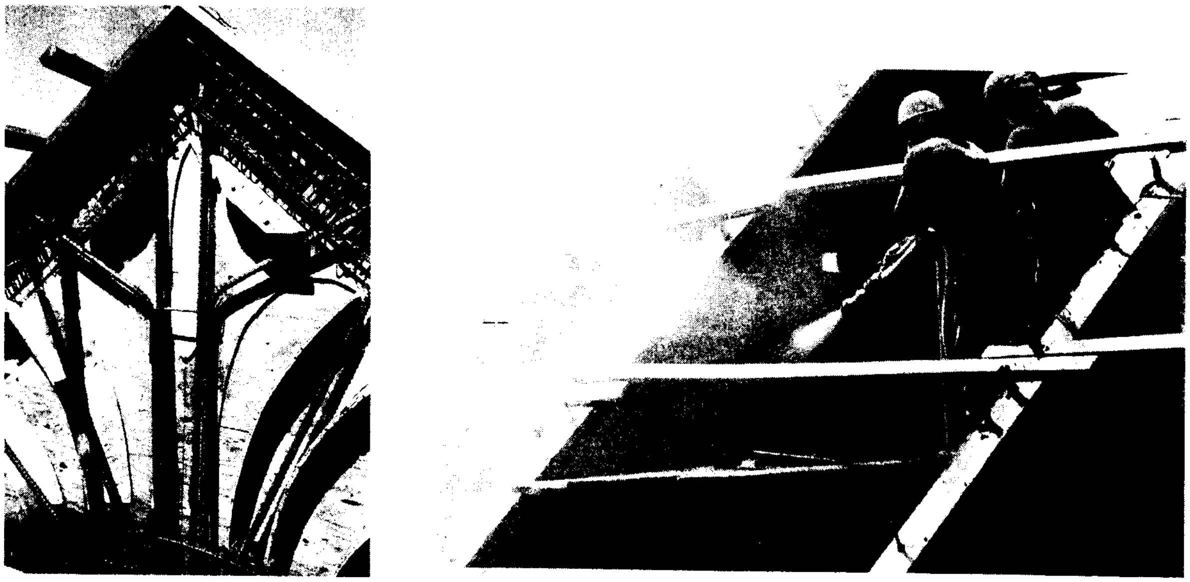

FIG. 8. Construction Sequence Involved Assembling Forms and Steel on Ground, Lifting Assembly into Position, and Placing Concrete Pneumatically: (a) Form Work in Place, Ready to Receive Concrete; (b) Gunite Being Shot into Forms

architectural demands, they cost substantially more than the cheapest standard roofing solution. The extra cost was paid for by reducing or eliminating finishes altogether. The walls were left unfinished concrete cinder block, (later painted by the owner) the floors were waxed concrete slabs, and the ceilings were fabricated from wood purlins (exposed on the inside and used previously in the forms) overlaid with construction-grade plywood sheets (also exposed on the inside). Reductions in the area of finishes paid for additional costs in the area of structure, so that although the structure cost more than conventional approaches, the overall cost of the building project was kept at the same level. This redistribution of the budget represents a departure from the norm of trying to design the structure to be built for the least possible cost. Instead of making the structure minimal and cheap, money was taken from other building operations and allocated to the structure, so that it could be exposed, commanding and respectable, integral with the space, and a major force in the architectural aesthetic of the project. Similar kinds of trade-offs would be generally possible in other building enterprises.

INTEGRATED DESIGN PROCESS

Leaving a building's structure exposed and using it architecturally to shape space places demands on the design team that are not normally encountered in conventional practice. Although these increased demands would apply to mechanical, electrical and environmental control professions, the greatest demands are placed on the architect and the structural engineer, and therefore the following discussion is limited to these two.

In current practice, it is generally the responsibility of the architect to focus on spatial and aesthetic issues while the structural engineer concentrates on making the building stand up. In a design project such as the trusses of the homeless shelter, the architect might make suggestions about the detailed geometry of the trusses, but the final structural design would be up to the engineer. Conversely, the engineer might propose changes to the architecture that would improve the structural behavior, but the final say about the appearance would rest with the architect. This process, which works fine for conventional building projects, does not work very well in cases where the aesthetics and the performance of the structure are strongly interconnected.

In the conventional design process, each professional makes their proposals within the confines of their own expertise and moreover tends to righteously guard their own professional turf. As a result, each one experiences difficulty when encroaching into the others so-called "realm," and the most creative solutions, which often lie precisely in the no man's land boundary layer between the two professions, are more difficult to bring into focus.

In contrast, it is useful to imagine the design process used by Herland at Westminster. We know that he built many models of the structure:

> When Hugh Herland . . . designed the Westminster Hall roof he did not look up textbooks on structural mechanics . . . His textbooks and stress diagrams were his innumerable models, which as we know occupied so much space that rooms in the King's palace had to be reserved for them (Waldram 1935).

For Herland's work on Westminster, all decisions concerning the design of the trusses, whether of a structural nature or an aesthetic one, would tend to be evaluated within the context of each. The proportions of the hammer beams to the great arch rib, or the hammer posts to the arch braces, would be measured on the basis of both structural integrity and visual harmony. In this regard, it can be said that each element is designed through a process in which both the structural efficiency and architectural beauty are considered on equal terms and integrated into a unified structure.

The process employed in the design of the homeless shelter's trusses was more similar to Herland's approach than to conventional methods, and is responsible for allowing the architecture and the engineering to work together towards creating a unified design. An abbreviated description of this process follows.

Structure That Supports Architectural Space

Initial structural forms were based on an analysis of spatial considerations. The first studies focused on the relationship between the bottom cords of the trusses and the space formed directly underneath (Fig. 9). During these studies, the writer and his colleague, Chris Alexander, made sketches of 14 different trusses, each with different arrangements of bottom cords. As a result of these studies, a truss design emerged in which the bottom cord formed three arcs, a low one at each end and a higher one in the middle. With this configuration, the room would be divided into three related spaces; a lower and more private one near the walls and windows, and a taller, more public one toward the center [Fig. 9(c)]. This particular arrangement appealed to the client as it provided spatial opportunities for newly initiated guests to feel secure in a public space. Although this design, obviously has aesthetic and formal references, it was selected primarily because it created a discernable and positive shape in the room.

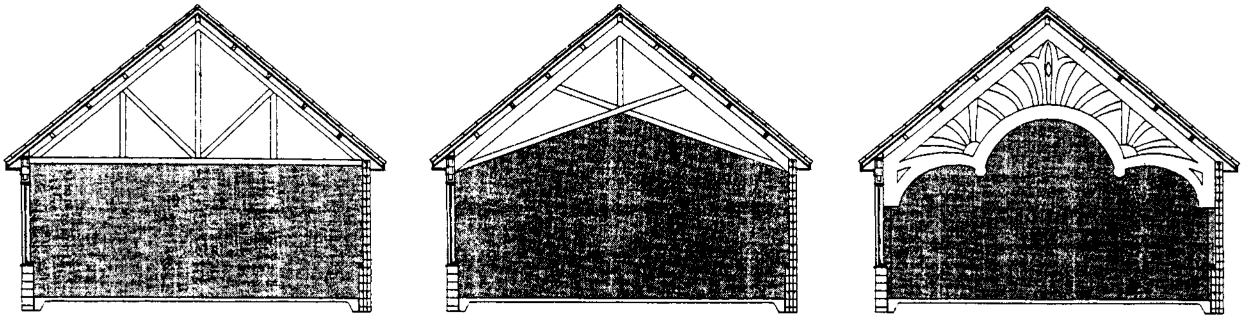

FIG. 9. Different Truss Designs for Homeless Shelter and Their Corresponding Spatial Qualities: (a) Conventional; (b) Scissors; (c) Trefoil

FIG. 10. Completed Truss (Photo © Mark Darley/Esto)

Once the basic planar geometry was fixed, a second investigation followed in which the spacing between the trusses was examined. These inquiries made use of physical models to capture the relationships between the overall depth of the truss and the spacing between them. If deep trusses are placed too close together they will obscure one another; conversely, if they are placed too far apart individual bottom cords will not occur frequently enough to define and shape the space of the room. As a result of these initial spatial analyses the general global forms of individual trusses and approximate spacings between them was determined.

Structural Efficiency

Engineers generally think of structural efficiency in terms of mathematical criteria with little consideration for spatial integration, and within this narrow definition, the proposal to use similar or identical structural forms for like spans may be seen as appropriate. However, the fact that a Fink truss is an efficient construction for intermediate span roofs does not necessarily