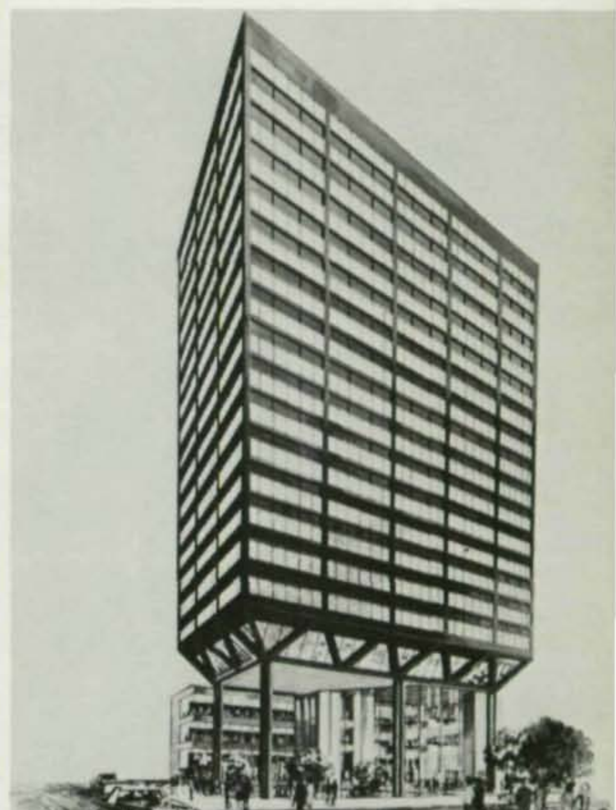
VOLUME XIV / NUMBER 1 / FIRST QUARTER 1974

MODERN STEEL CONSTRUCTION



MODERN STEEL CONSTRUCTION

Published by

American Institute of Steel Construction

101 Park Avenue, New York, N. Y. 10017

VOLUME XIV / NUMBER 1 / FIRST QUARTER 1974

CONTENTS

1974 T. R. HIGGINS LECTURESHIP AWARD

Dr. Joseph A. Yura has been named recipient of the Fourth Annual T. R. Higgins Lectureship Award. Dr. Yura was chosen to receive the $2,000 award for his contribution to the fund of engineering knowledge based upon his paper entitled "The Effective Length of Columns in Unbraced Frames" (AISC Engineering Journal, April 1971.)

The award will be presented at the 1974 National Engineering Conference banquet on Thursday evening, May 2, in Chicago.

1974 PRIZE BRIDGE COMPETITION

Entries are invited for the 46th Annual Prize Bridge Competition to select the most beautiful steel bridges opened to traffic during the calendar year 1973.

The members of the 1974 Prize Bridge Jury are:

William S. Allen, FAIA *Anshen & Allen, San Francisco, California*

David G. Hammond, F. ASCE *Daniel, Mann, Johnson & Mendenhall, Baltimore, Maryland*

Craig P. Hazelet, Hon. M. ASCE *Hazelet & Erdal, Louisville, Kentucky*

William M. Sangster, F. ASCE *President-elect American Society of Civil Engineers; Director, School of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia*

Reece H. Wengenroth, F. ASCE *Executive vice-president, Westenhoff and Novick, Inc., Chicago, Illinois*

Entries must be postmarked prior to May 25, 1974 and addressed to the Awards Committee, American Institute of Steel Construction, 101 Park Avenue, New York, New York, 10017.

1973 ARCHITECTURAL AWARDS WINNER
COLLEGE OF DuPAGE -- INSTRUCTIONAL UNIT ONE

In the recently issued 1973 Architectural Awards of Excellence booklet, the steel fabricator and steel erector of the award-winning College of DuPage structure should have been listed as:

Steel Fabricator:
Mississippi Valley Structural Steel
Division of Debron Corporation
St. Ann, Missouri

Steel Erector:
Kenwood Construction Company
Hinsdale, Illinois

What's UP in KANSAS CITY?

"Prime Time," Kansas City, Mo.'s major downtown revitalization program, started in 1971 and targeted for completion in 1985, has added an "ahead of the times" design to its continuing parade of construction projects. The Mercantile Bank Tower is destined to establish some new directions for Twentieth Century architecture.

Filled with unusual structural and engineering concepts, the Tower will incorporate at least three major build-

Artist's rendering of 20-story Mercantile Bank Tower, Kansas City, Mo.

ing innovations — employed in Kansas City for the first time — which will set it apart from nearly every other high-rise structure in the country.

- The building will feature heat-shielded exposed steel girders which perform a dual role of framing and enclosing.
- It will contain a steel space truss, reminiscent of the truss system used in bridge construction, to transfer the weight of the 16 upper floors to the main support columns and the elevator core.
- It will be supported by five massive steel columns, each approximately 60 ft in height, and the elevator core. These columns will be filled with a solution of water and antifreeze to provide fire protection for the columns.

The heat-shielded girder system, developed by U.S. Steel Corp., was introduced in a 55-story high-rise office structure — One Liberty Plaza — completed in New York City in 1971. The system not only incorporates a novel fire protection method, but it enforces a dual function upon the structural beam.

In conventional steel column and beam construction, the structural frame is fireproofed and then the masonry or metal curtain wall enclosure is added. This is not so in the Mercantile Tower.

The exterior horizontal girders serve two functions in the Mercantile project. They are structural components, forming part of the framing system and replacing the more conventional concealed spandrel beams required to carry floor loads. Also, these spandrel girders, acting with the exterior columns, resist all the wind forces on the tower. Since they are exposed, they also act as 50 percent of the building's outside wall.

Fire Protection

As structural members, the spandrels must be protected against fire. In the Mercantile project, fire protection material will be applied to the interior face of the spandrel in sufficient amounts to meet the Kansas City building code. Special deflectors welded on the exterior of the girder will direct flames away from the web of the girder — in case of fire starting in the building.

The exterior face of the spandrel will be painted, both to decorate the facade and to weather protect the steel.

"This structural solution," says Philip Prince of Harry Weese & Associates, project architects, "is also an architectural solution. Nearly every piece of exposed steel functions to make the total structure work. Most of the building wall requires no other materials to decorate, protect, or cover it. Our structure is exposed to full view — no embellishments, no false skin.

"By adopting this design solution, we are not only able to create the boldest of steel structures, we are able, as well, to achieve measurable economies, since less finishing material will be required. Because the girders in the structural framing system also function as the exterior curtain wall, a secondary enclosure system was not required."

Steel Space Truss

A second major structural innovation in the Mercantile project is the steel space truss. Located at the top of the five support columns, about 60 ft above a sunken plaza level, the truss — comprising giant vees inclined outward at a 45° angle around the four sides of the building — will be one of the building's most prominent visual features.

The principal function of the four-sided steel truss is to transfer the weight from 24 columns in the top 16 floors to the five main columns and the elevator core. Since the engineering solution did not require a full space frame, the truss system will create a large, unobstructed area for housing the building's mechanical equipment.

Because of the truss transfer system, the space at the base of the building will be left open to provide maximum accessibility to pedestrian traffic.

Water-filled Columns

The third innovation — and perhaps the most dramatic — concerns the building's massive support columns. There are to be five of these columns, each fabricated from four rolled wide-flange steel sections welded together into the shape of a hollow cross.

All columns will be filled with a solution of water and antifreeze which provides an advanced and novel means of fire protection, the first time that such a system has been used in Kansas City.

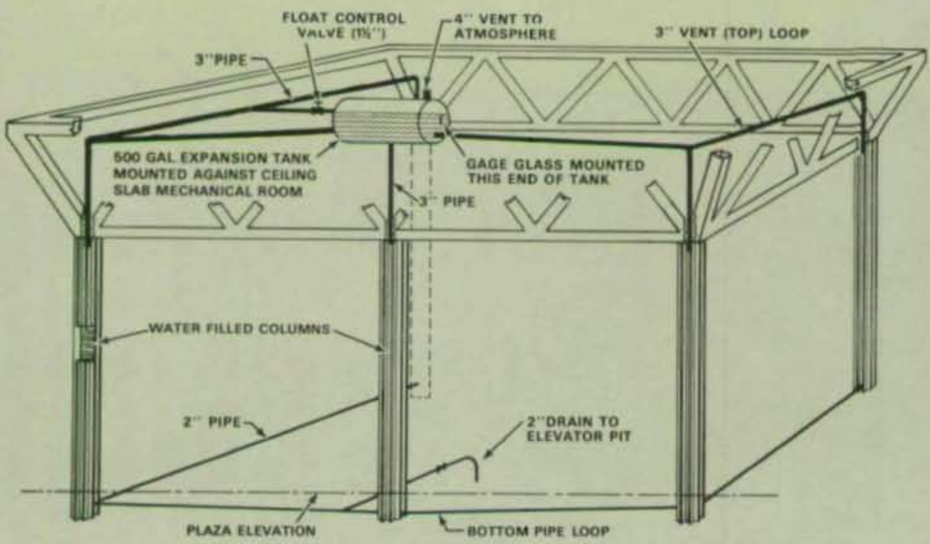
This water-filled-column method fire protects not only the exposed surface, but also the interior. The water-filled columns — containing about 8,700 gallons of liquid — will reach to the steel space truss approximately 60 ft above the plaza. Each column will hold about 1,740 gallons of solution. To accommodate any volume change in the solution, an expansion tank with a piping loop connecting the tops of the columns will be placed inside the steel space truss which encloses the mechanical equipment floor.

"The tank will provide space for expansion of liquids in the columns in hot weather — or in the unlikely event of a fire — when the liquid can be expected to expand." points out Dan Duncan, project engineer of Jack D. Gillum & Associates, structural engineers on the project. "In case of fire the exterior columns must continue to provide structural support. They must, therefore, be protected. Rather than conventional fire protection — such as cladding the entire column with steel plate over a layer of fire-retardent material — the principals decided to fireproof the columns with water from within. Not only was this an unusual — and attractive — design and engineering solution, it was both practical and economical."

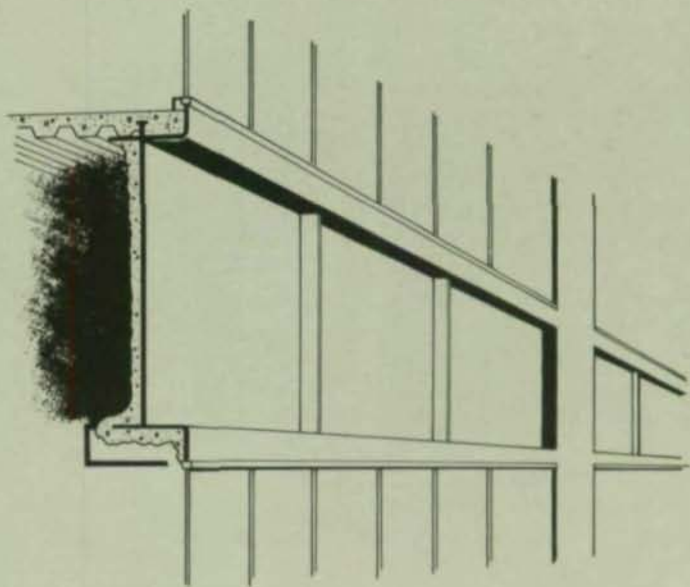
Advanced Steel Structure

The 20-story structure will comprise 244,000 sq ft of space. General offices for the bank will be housed in a three-story mini-building of exposed aggregate precast concrete tucked under the space frame. About 2,200 tons of structural steel — A36 carbon steel and A572 high-strength, low-alloy steel — will be needed for the total project.

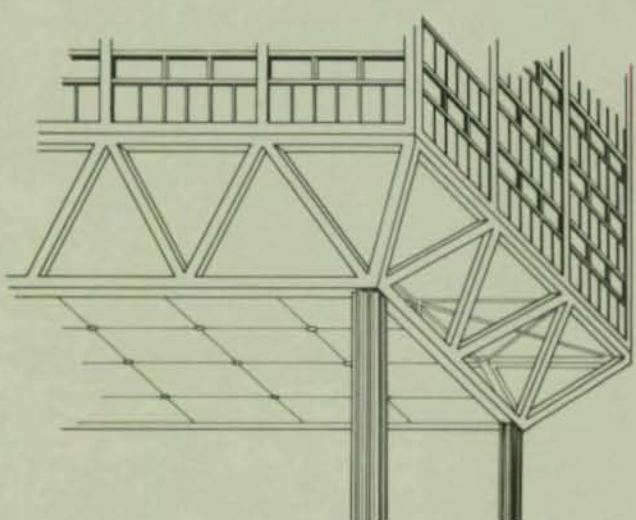
The building is scheduled for completion later this year.



Water-filled columns fire protect not only the exposed surface, but also the interior.



The exterior face of the spandrel will be painted, both to decorate the facade and to protect the steel from the weather.



The steel space truss comprises giant "vees" inclined outward at a 45° angle around the four sides of the building.

Architect:
Harry Weese & Associates
Chicago, Ill.

Structural Engineer:
Jack D. Gillum & Associates
St. Louis, Mo.

General Contractor:
Concordia Management Services Limited
Kansas City, Mo.

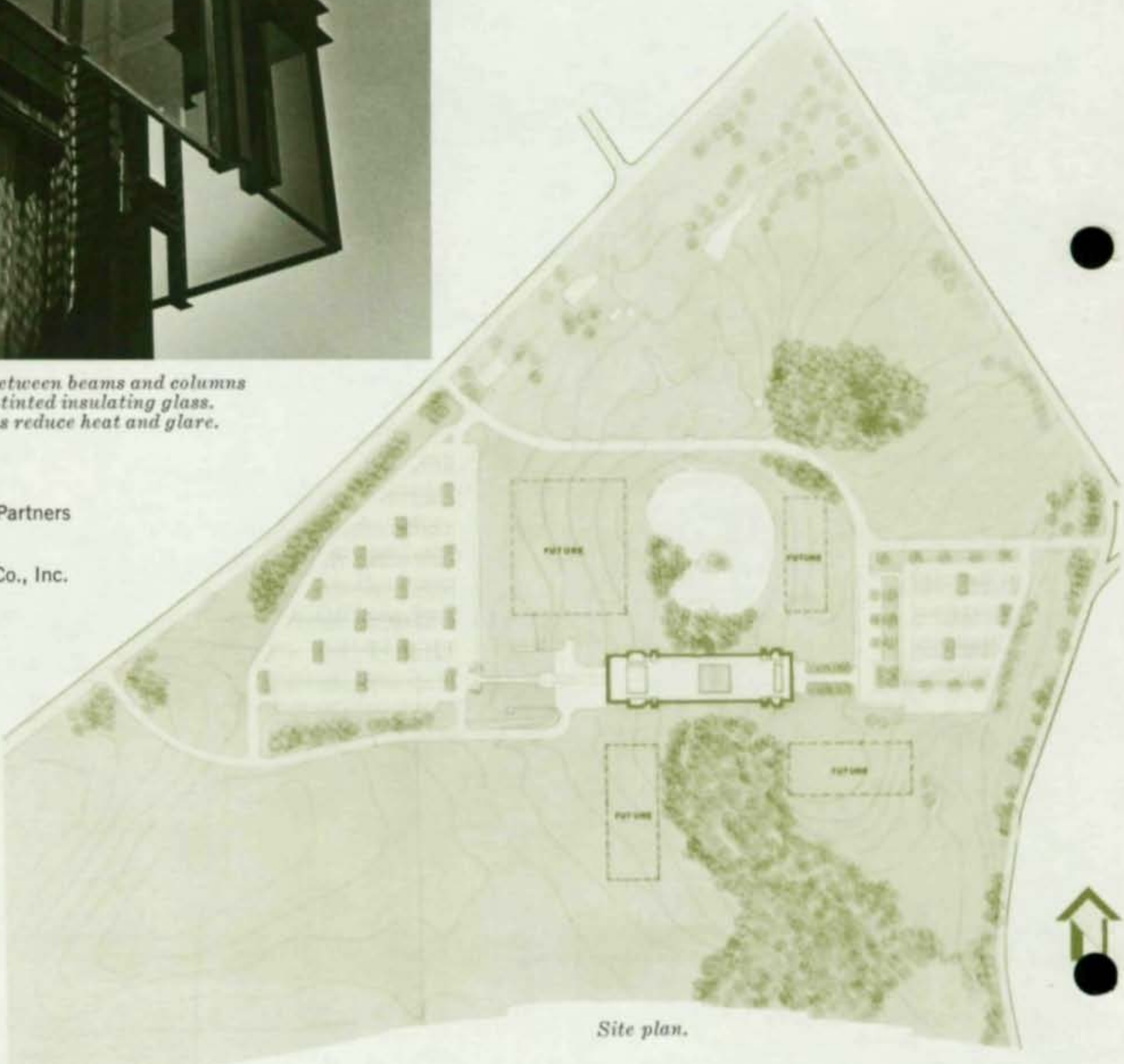
Exterior wall space between beams and columns is glazed with bronze-tinted insulating glass. Solar glass sunscreens reduce heat and glare.

EXPANSION INSURED

Architect:
Vincent G. Kling & Partners
Philadelphia, Pa.

Structural Engineer:
A & R Engineering Co., Inc.
Philadelphia, Pa.

General Contractor:
L. F. Driscoll Co.
Bala Cynwyd, Pa.



Site plan.

Weathering steel exterior columns and spandrels blend with the building's wooded setting.

The National Liberty Life Corporation, Frazer, Pa., having outgrown its present quarters, acquired a 92-acre site for expansion. Projected growth of the insurance company's business indicated a need for 135,000 gross sq ft of office building space to accommodate 515 to 550 employees by 1975, in executive, marketing, operations, and computer departments.

An in-depth study of corporate philosophy, goals, functions, and paper work flow resulted in a recommendation that executive-marketing offices be grouped together in one cluster and operating areas be arranged around a computer center in another cluster, with a separate structure for the power plant.

The new headquarters is a four-story structure, 360 ft x 100 ft. The central section bridges a valley with a small stream. The executive marketing area, at the east end of the building, centers around a lobby and reception area. At the west end is the operations area with larger, more open spaces centering on computer and mail facilities. Both areas are designed for future expansion.

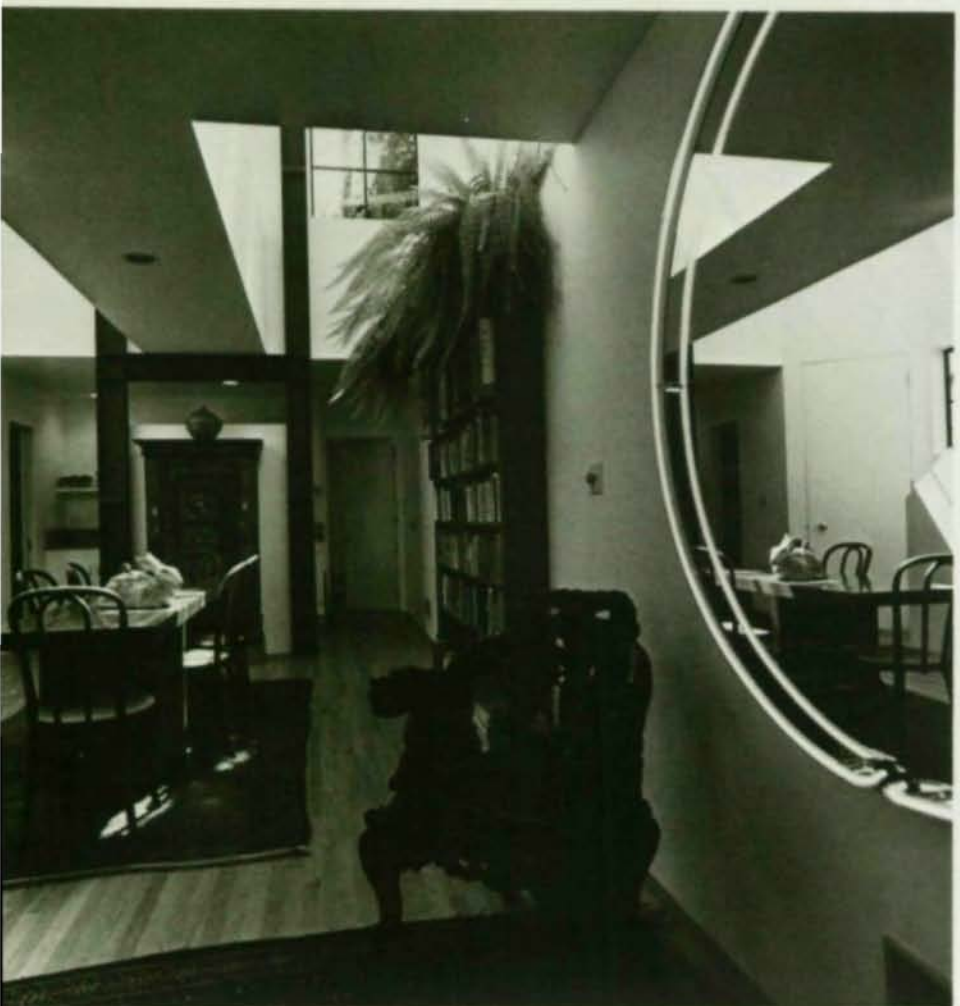
The result is a unified, functionally efficient building, allowing maximum growth flexibility for the future without weakening the initial unity, because the valley — the strong, natural environmental factor — is maintained as a north-south axis perpendicular to the structure. On both sides of this natural axis, additions can be constructed.

The steel-framed building has weathering steel exterior columns that blend with the building's wooded setting. As the bare steel weathers to a rich, dark brown, it develops a self-protecting, natural oxide coating.

Exterior wall space between beams and columns is entirely glazed with bronze-tinted insulating glass, except that stair enclosures and service cores have walls of red-earth colored brick. On the east, south, and west facades, solar glass sun screens reduce the summer heat gain, without sacrificing the views of the surrounding country.

The structure has a 52-ft wide central bay, framed on each side by two 20-ft bays. Fluorescent lights on 5-ft centers are used in office areas; incandescent lighting is employed in executive areas, corridors, lounges, lobbies, and vending spaces. Energized cellular steel deck underfloors are used for electrical, telephone and intercommunication systems. Together with the 5-ft lighting module, this system permits complete flexibility in desk and partition location. Partitions are demountable laminated gypsum walls. Floors, except in computer, wash room, and stock room areas are carpeted. Ceilings are of acoustical tile.

STEEL AT HOME in TOW



Steel frames were left exposed in both units of San Francisco townhouse.

Imaginative applications of steel continue to show up in residential construction. Whether an architect is faced with revamping a century old flat on a city street or a difficult configuration, such as a steep hillside, steel has proved its versatility as well as practicality as a construction material for homebuilding.

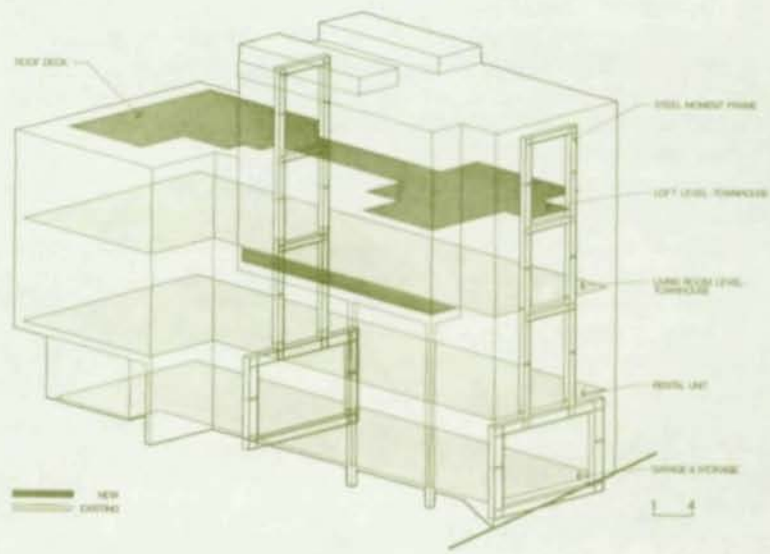
Here is a study in contrasts of how steel served aesthetically and functionally for two homeowners.

Converted Townhouse

In San Francisco, an architect turned a small 19th century two-unit flat into a two-story town house, a rental unit, and garage. A fourth story was added to the building to meet contemporary seismic and wind load requirements.

Since the main section of the building only measures 12'-0" wide, steel rigid frames were necessary to resist overturning moment due to wind load. Shop-welded and field-bolted frames were threaded through the existing structure. The rigid frames take the place of wood shear walls, permitting the town house openness in both plan

Isometric view of San Francisco townhouse.



SAN FRANCISCO RESIDENCE

Architect:
Daniel Solomon, AIA
San Francisco, Calif.

Structural Engineer:
Hirsch & Gray
San Francisco, Calif.

General Contractor:
H. D. Grae
Mill Valley, Calif.

N or COUNTRY

and section with a maximum exposure to sunlight and view. The exposed frames are painted tile red.

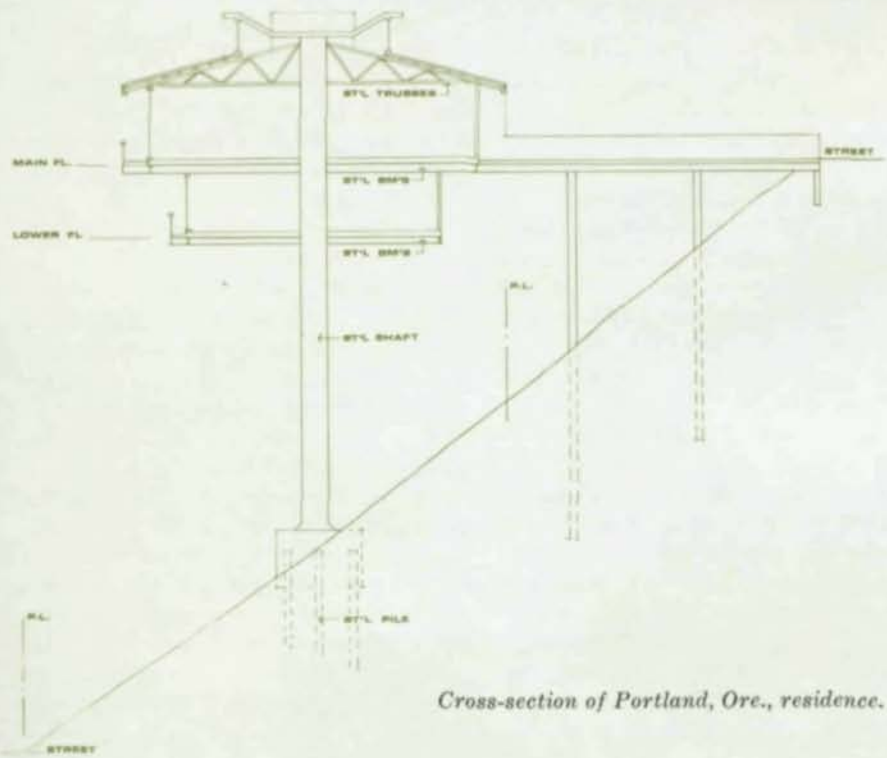
Hillside Home

An extremely steep site (approximately 40°) in Portland, Ore., with streets abutting both the high and low sides, created a shallow building envelope to work within. Because the property line at the high side is 30′-0″ from the street and the grade drops so quickly, a 36″ diam. circular steel column was chosen to support the structure for both drama and simplicity of structural origin.

The center column, erected as a single support with radiating steel roof trusses, supports a series of main floor beams, from which the lower floor beams, in turn, are supported. From this point, the remainder of the construction is wood, which is synonymous with the locale.

The main floor plan consists of living, dining, and kitchen spaces, along with master bedroom with bath and sauna facilities, a den and powder room. The lower floor is for guest use.

In the Portland, Ore., residence a circular steel column supports the structure.



Cross-section of Portland, Ore., residence.

PORTLAND RESIDENCE
Architect:
Zaik/Miller/Butler, AIA
Portland, Ore.
Structural Engineer:
Arthur James Engineers, Inc.
Portland, Ore.
General Contractor:
Barnard & Kinney, Inc.
Beaverton, Ore.

First Vehicular Cable-stayed Bridge in the U.S.

by William L. Gute, Partner
Gute & Nottingham, Structural Engineers
Juneau, Alaska

The site of the first cable-stayed vehicular bridge in the United States is Sitka, on Baranof Island, in the southeast panhandle of Alaska. The bridge replaces a small ferry to Japonski Island, site of a hospital, residential area, a Bureau of Indian Affairs boarding school, and a recently completed jetport. The close proximity to the picturesque harbor and town of Sitka spurred an intensive effort to make this large, high structure as unobtrusive as possible.

Why is this bridge significant? As the first vehicular cable-stayed bridge in the U.S., it shows this type design is both economical and practical in this country. The cable-stayed bridge competes directly with truss bridges in the intermediate span range. Shorter structures are usually girder bridges; longer structures, suspension bridges. Both girder and suspension bridges can be made beautiful. But most truss bridges — i.e., bridges used in the intermediate-span range — are ugly. The U.S. bridge designer now has another design to choose from for intermediate-span bridges, a more aesthetic design. In the years ahead, more and more cable-stayed bridges are likely to be built in the U.S. — because of competitive cost, clean details, and aesthetics. The cable-stayed box-girder scheme satisfies the requirements of this site better than any other bridge studied. It has these advantages:

• The cable-stayed bridge is the **most economical** alternate. It requires only standard construction techniques — no special erection and deck surfacing problems, as with orthotropic design.

• The bridge **blends in** harmoniously with the surroundings. Shallow girders look sleek in profile. Cables are small, and free-standing pylons clean. Underdeck details are also very clean. Box girders require minimum bracing and all stiffeners are inside. With waterfront views of antennas, masts, trolling poles, piling, and trees, this bridge accentuates the waterfront mood.

• In the center of the span, no superstructure extends above the deck. Thus no obstruction to seaplanes.

• The structure will be **easy to maintain.** Cables are galvanized and all other steel sections are closed boxes with few places for corrosion.

Used in Europe

The cable-stayed girder bridge has been widely used in Europe — but has not caught on in North America. The Germans have used single and multiple stays in various configurations, including the bundle, fan, harp and star systems. These have been arranged in three different planes: vertical exterior, vertical interior, and sloping from the top of an A-frame tower to an exterior anchorage.

Why hasn't the cable-stayed bridge been used in the U.S.? Several reasons. The cable-stayed bridge is highly indeterminate, difficult to analyze exactly. Most cable-stayed designs have considerable flexibility in the deck, more than permitted by AASHO specs. And until recently, most U.S. bridge designers, with some exceptions, haven't been very concerned with bridge aesthetics. Also contributing to the lack of use of the cable-stayed bridge is lack of knowledge about cost and design, the difficulty of getting a new bridge-type approved through government bureaucracies, and a general inertia to change.

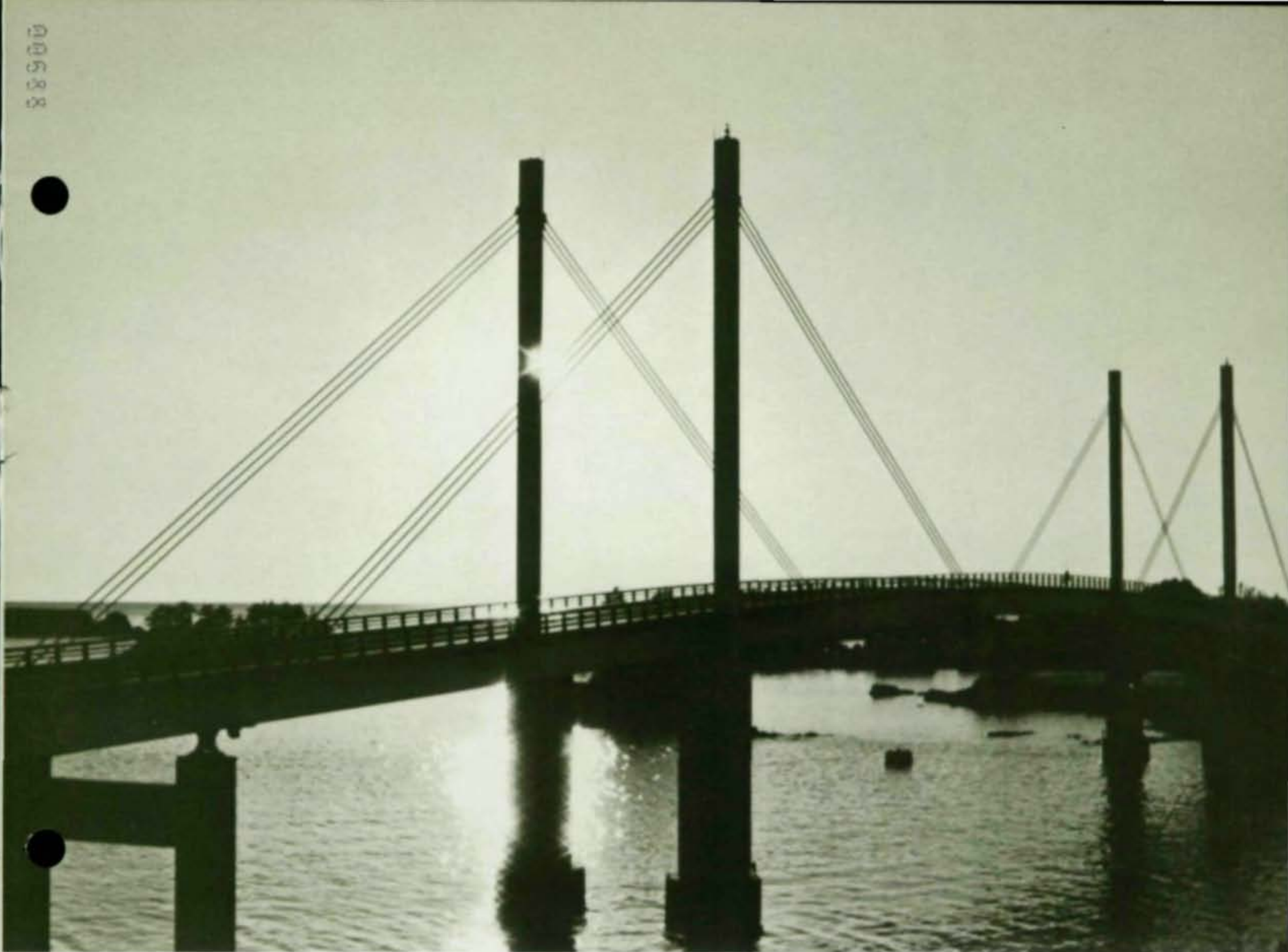
Two modifications are necessary to adapt German designs. First, bridges must be made stiffer. Live load deflection of the Severn Bridge at Cologne is 1/225 of the span — 3½ times more flexible than allowed by AASHO specs. Second, cables must be more adequately protected from traffic and road salt corrosion. This means anchoring them beyond the edge of the deck.

Very little road salt splashes on the cables because they are far enough away from the deck edges. The cables are galvanized to provide corrosion resistance. There is so much rain in Southeast Alaska that salt is washed off the cables anyway.

The U.S. is too conservative on live-load deflection allowances in the deck, especially on longer spans. Permissible deflection should be based on a structural-response criteria for vehicle safety and passenger comfort — not a ratio of span lengths, as presently. There is a real need to revamp specifications here. The Sitka Harbor Bridge achieves the U.S. requirements on stiffness by anchoring the backstays over the approach piers, and by using steeper cable angles than in most German designs.

It is most important to provide for cable adjustments in the field. Reason? To insure that the girders are at the proper elevation, and the pylons vertical in the finished bridge. It is tempting to use cables with open sockets on each end because of the simplified details; but fabrication and construction inaccuracies and the variable modulus of elasticity of the individual cables pre-

Based on an article in the November 1973 issue of CIVIL ENGINEERING—ASCE, official monthly publication of the American Society of Civil Engineers.



clude this. The camber of the girders is the most difficult thing to fabricate exactly; any change from theoretical changes the cable stress — and hence length. Cable stress also changes if bearings are not set at exact elevations; or the base of the pylons is not exact. In short, the cable adjustments enable you to nullify unavoidable fabrication and erection errors.

Superstructure Design

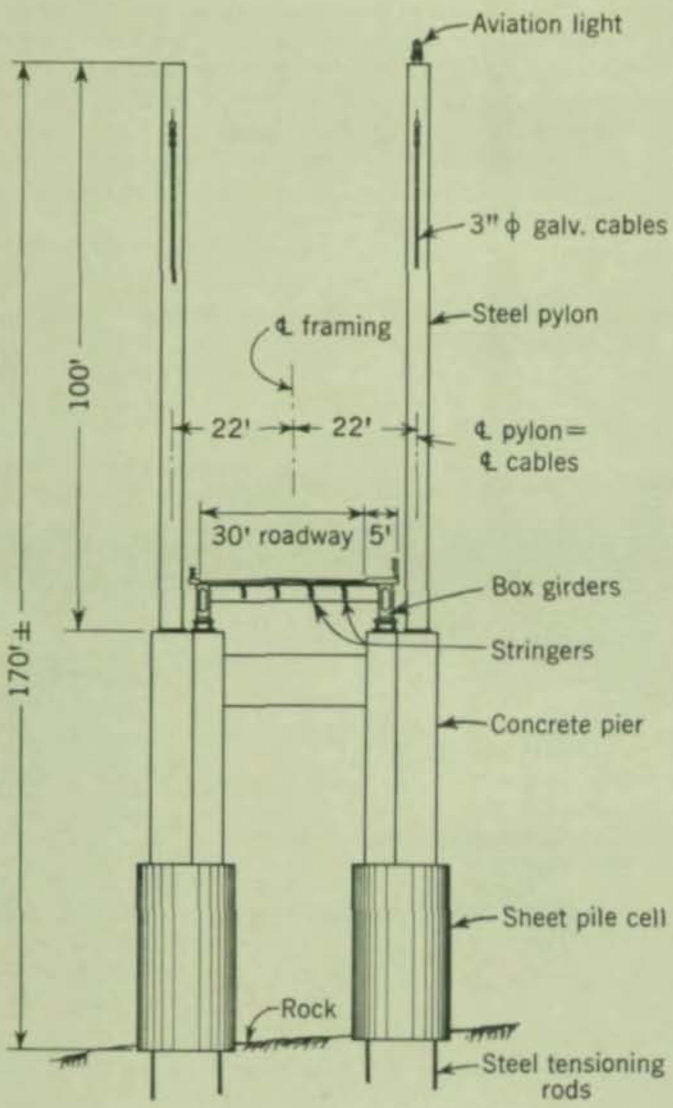
The same floor and girder system is used for the full 1,250-ft length of the bridge. The bridge has four 125-ft approach spans, two 150-ft side spans, and the 450-ft main span. The approach on the Sitka side is a horizontally curved continuous section; the one approach span on the Japonski Island side is simply supported. The side and main spans are continuous with expansion joints between the approach and side spans. The result is a constant depth roadway — i.e., a constant depth girder — from one end of the bridge to the other.

Single-Stay Cables

A **single-stay** system with attachments to the girders at the third points of the 450 ft main span is used. Reason? For simplicity and to minimize the number of deck cable anchors, which are **very** expensive. Multiple stays are feasible only when they anchor **directly** to the girders. The back stays are anchored over approach piers to minimize live load deflections. If this is not done, computations show, the pylon-cable-girder system acts like a flexible girder haunch — the structure is too flexible. The pylons are free standing vertical welded steel boxes, 3 ft x 4 ft in section, fixed to the piers with high strength threaded rods anchored in the concrete. The side plates of the boxes are ½-in. thick; and the end plates are 1½ in. thick in the lower part, and ¾-in. thick in the upper part of the pylons.

Cables are attached with open sockets to large plates at the tops of the pylons. This eliminates cable saddles (i.e., an assembly on top of the pylon over which cables pass) and simplifies details. The strong axis of the pylons is perpendicular to the roadway to resist wind forces. The tops of the pylons move about 2 in. in the plane of the cables due to live load deflection.

Height of pylons is determined by the optimum angle of the cables. The controlling factor — cable strength or live load deflection — depends on the angle the cable makes with the horizontal. Plots of cable stress and vertical stiffness yield an optimum angle of 26.5° for a level structure. For smaller angles, deflection controls design; for



Cross-section of bridge deck and supporting piers.

larger angles, cable stress controls. Pylons, about 100 ft high, have extra height above the cable-attachment points for aesthetic reasons.

The tension in each stay is about 1000 kips. Three 3-in. bridge strands were used for each stay; thus conventional galvanized strand and fittings could be used. A factor of safety of three on the breaking strength of the cables is used. Theoretically, then, only one cable of the three-cable group is required to support the full dead and live loads.

Cable Anchors

The cable anchors to the deck were studied more carefully than any other structural detail. They act as cantilevers with the plane of the cables 5.75 ft from the center of the girders. They must: be stiff enough to ensure interaction between the two box girders; possess great strength in the plane of the cables; support intermediate longitudinal stringers; pass through the relatively shallow box girders; and minimize stress raising details at the intersection of the girders.

The anchor beam consists of a 5-ft diam. welded steel tube with 1-in. thick walls. The circular section exhibits equal bending strength in all directions, eliminating the problem of cable orientation. The tube is stiffened with ¾-in. diaphragms at the girder webs and cable sockets. The cables themselves are anchored through steel tubes welded through the anchor tube, with an anchor socket and spanner nut. Cable adjustment is provided at this point. The entire anchor tube was shop assembled, complete with box girder and longitudinal stringer stubs welded in place. High-strength bolted field connections splice the box girders and stringers on each side of the anchor.

Girders and Deck

A two-girder deck system is used to minimize the bending moments in the deck cable anchor beams. The two girders are close to the ends of the anchor beam and to the cables; drawing moment diagrams, it can be seen that the moment for the two girders is lower than if a series of girders were used to support the bridge deck.

The girders are box sections. We used box girders — rather than plate girders — because of their resistance to compressive stress, resistance to corrosion, and ease of erection. Plate girders score poorly on these three counts. In the U.S. today, box girders are being used more and more. The Sitka Harbor girders are unusual in that they are under high compressive stresses, stresses introduced by the horizontal component of the tension in the cables. The box girder was the logical choice here, for it is stiff; it makes a good column. The plate girder, on the other hand, acts "like a piece of spaghetti" until a lot of bracing is added. Many plate girders require a temporary horizontal bracing system to stiffen the compression flange during erection.

The box girder has better corrosion resistance than the plate girder, because all the stiffeners are inside and the web and flange plates are exposed to the weather on only one side. Most corrosion starts around welds and stiffeners. Here, they are all inside the box and out of the weather.

The girders contribute only 10 to 15% of the stiffness of the main span, the cables providing the balance. Thus, even doubling the stiffness of the girder would only increase the span stiffness by a small amount. The girder depth was selected at 1/25 of the side-span length, or 6 ft (this 1/25 is a commonly used rule-of-thumb). This is 1/75 of the main span. A girder depth of 1/100 the main span was used for the North Bridge at Dusseldorf, Germany, which also has its backstays anchored over approach piers. The box girder on the Sitka Harbor Bridge really spans between deck cable-anchor beams; this is a distance of about 150 ft, the same length as the side spans. The width of the box girders is 30 in. All field splices are made with ⅞-in. ASTM A325 bolts.

A conventional reinforced concrete deck slab, composite with the girders, simplifies erection and adds stability to the main span. The interior of the deck is supported on small longitudinal stringers spanning between floor beams on 25-ft centers.

The dead load deflection at the center of the span is about 25 in. The girders are so ineffective in contributing to stiffness that the strains in the girders, and hence the stresses, are about the same no matter how thick the flanges are. Greater girder depths will tend to increase the strains and hence the stresses.

One method of handling these large moments is to provide temporary hinges over the piers and at the cable anchors until the full dead load has been applied. The hinges could then be locked up for improved performance under live loads. This method is impractical, however, because the joints are at the points of high live load moment and the details would be costly.

Better solution? To camber the girders (convex curve facing upward) so that when the heavy concrete deck is poured, the girders will deform to the proper roadway grade. Cambering does not eliminate deflection. It merely makes allowance for it so the roadway comes to proper grade after the deflection due to the weight of deck concrete has taken place. This method increases cable stress and complicates the erection procedure, but it is used in European designs. With enough cable attachment points and a shallow enough girder, virtually any moment diagram can be attained by cambering.

The girders in the Sitka Harbor Bridge are merely cambered to mirror the dead load deflection, the same as any other girder bridge. ASTM A514 steel, which has a yield stress of 100 ksi, is used to accommodate strains in high moment areas; this greatly simplifies details and erection procedures. Larger bridges will probably have to use some method of cambering to reduce girder strains. The remainder of the structural steel is A572, with A36 used for minor low stressed members.

Alternate Designs Considered

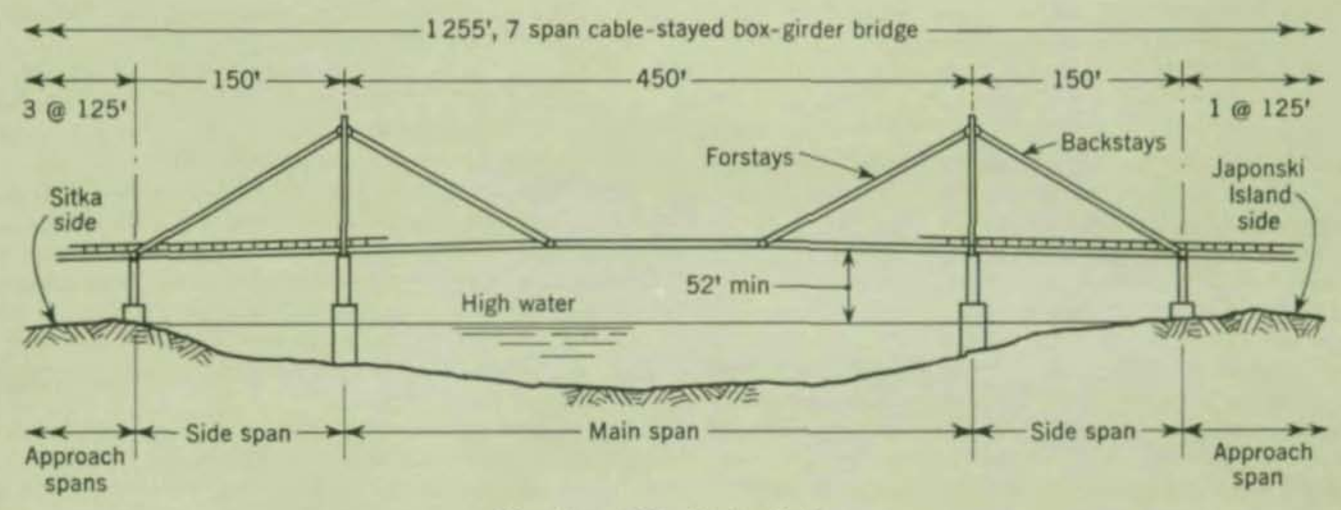
Six different bridge schemes were considered for this crossing.

Scheme	Description	Cost Ratio (Cable Stayed Girder = 1.00)
I	Plate Girder w/Fenders	1.15
II	Plate Girder	1.13
III	Orthotropic Box Girder	1.04
IV	Through Tied Arch	1.04
V	Half Through Tied Arch	1.06
VI	Cable Stayed Box Girder	1.00

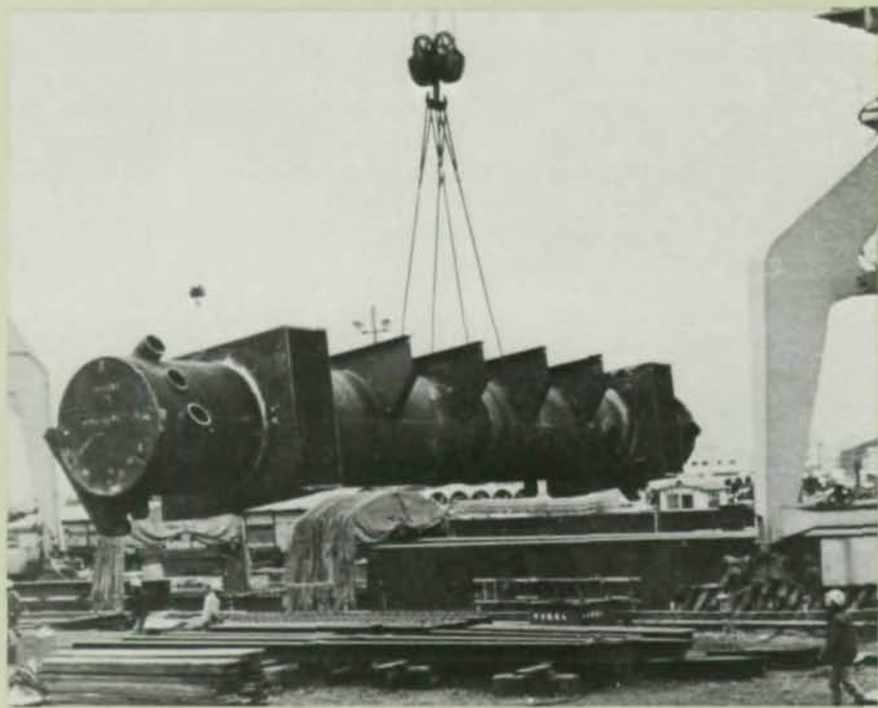
Note the 250-ft main span of Scheme I must be skewed to accommodate the fender system along the sides of the 200-ft navigation channel. Both the fender system and main piers would be in about 52 ft of water, making this the most expensive scheme. The fender system would also be difficult to maintain and would be an eyesore, especially at low tide. A main span of about 450 ft would move the main piers out of the deep water and place them beyond the limits of the channel defined by a navigation light on one side and the face of a large dock on the other, thus eliminating fenders.

The next two bridge schemes studied had spans of 300 ft — 450 ft — 300 ft. These spans were too long for Scheme II, the continuous plate girder, and its cost is high. The orthotropic deck box girder, Scheme III, is only about 4% higher than the cable-stayed scheme, but it shares one major shortcoming with Scheme II: it is too deep at midspan. Either scheme will have a superstructure depth of about 14 ft at midspan. As a comparison, the 6-ft depth for the tied arch and cable stayed schemes results in an 8-ft reduction in grade line. This is, of course, reflected back into the approach fills and results in lower quantities, less room required for the roadway section, and improved visibility from town. The resulting savings are not included in the cost comparisons, but would give the latter alternatives a decided advantage.

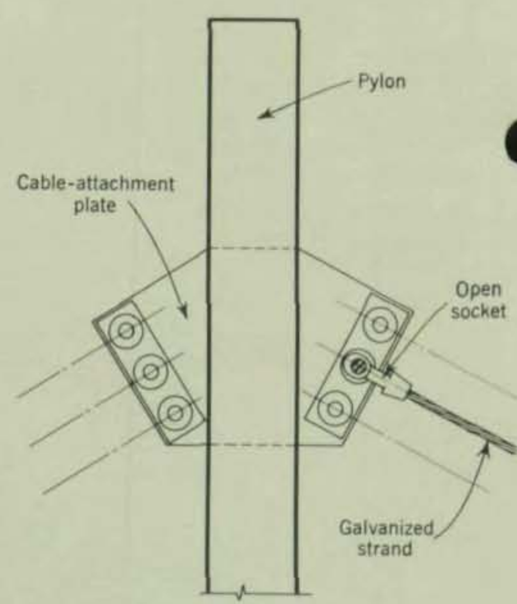
A through truss would have a low grade line, but any type of simple or cantilever truss design is out of the



Elevation of Sitka Harbor Bridge.



The tubular cable-anchor beam was shop fabricated with box girder and longitudinal stringer stubs in place.



Cable attachment details near top of pylons.

question for this site because of aesthetics and maintenance. The bridge as viewed from Sitka will appear foreshortened. The many small members and connections would be a maintenance headache in the seacoast atmosphere.

The remaining three schemes studied make use of small piers to reduce the side spans to 150 ft. The one on the Sitka side is founded on a rock outcrop about El. +5 and the Japonski one is at about El. +6. These piers are therefore quite short and inexpensive.

Schemes IV and V, the through and half through tied arches, received close scrutiny. The through arch looks good from the side, but the usual view will be foreshortened. The harbor is used for seaplane operations and the high part of the superstructure is in the center of the channel. The overall view of the bridge does not blend with the site as well as the cable-stayed scheme. The price of the through arch is virtually the same as the cable-stayed scheme.

The half through tied arch, similar to the Port Mann bridge in Vancouver, B.C., is probably the strongest competitor of the cable-stayed scheme at this site. Half of the arch rise is carried beneath the deck, which lowers the piers and decreases the amount of superstructure above the deck. The floor system can be made quite shallow. It is good looking when viewed from the side. The cost is only about 6% higher than the cable stayed.

It has disadvantages, however. The bracing between the ribs and in the sway frames at the piers will normally appear foreshortened. The low elevation of the steel combined with the many members and connections will encourage corrosion. The skew of the crossing may limit the optimum location at the ribs beneath the deck because of interference with the limits with the navigation channel.

Bridge Substructure

The bridge is supported on four piers in the water and two land piers. The upper part of all piers is a reinforced concrete frame about 40 ft high. The frame legs are 4 ft x 7 ft in sections, except for the piers supporting the pylons, which are T-shaped, 6.5 ft x 13.5 ft in outside dimension. The land piers are supported on short H-piles driven to bedrock.

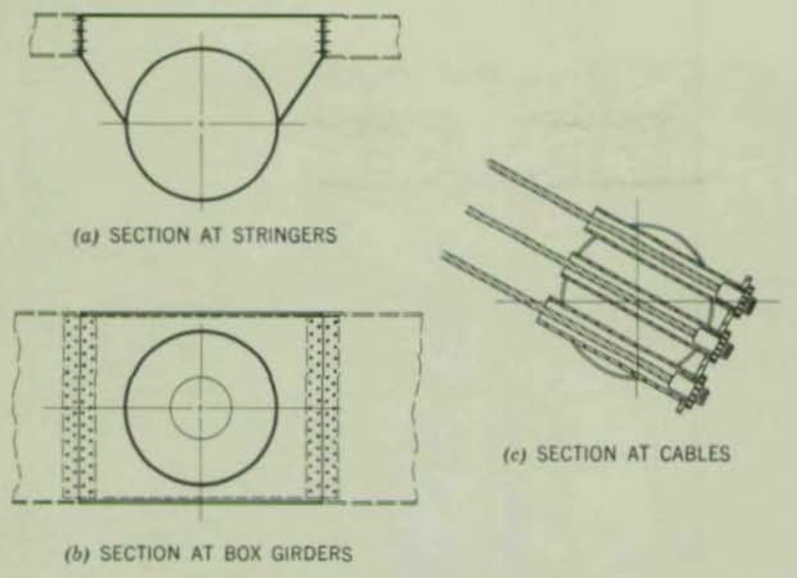
Conventional concrete deteriorates rapidly when exposed to seawater in Alaska. The high, cyclic pore pressure from the tides, combined with weather producing many freeze-thaw cycles, tends to pop off weakened concrete. To prevent this, piers are supported on concrete-filled steel sheet-pile cells. The cement used in the cell concrete is Type II for improved resistance to seawater. These cells are 17.5 ft in diameter and 35 ft high for each leg of the main piers; and 12 ft in diameter and 15 ft high for each leg of the other two water piers. The tops of the cells are 1 ft above the highest tide.

Sheet-pile steel is specially formulated to resist salt water corrosion. In addition, sheet piles have two coats of coal-tar epoxy and a galvanic cathodic protection system to make them as maintenance free as possible.

The upper part of the cells is heavily reinforced to distribute the pier loads over the full area of the cell. This heavily reinforced concrete, the lightly reinforced remainder of the cell concrete, and the bedrock are tied together with twelve 50-ton prestressed rock anchors per cell drilled and grouted 30 ft into bedrock. These anchors provide improved factors of safety against overturning or sliding under earthquake forces.

Constructing the Bridge

Temporary wooden work bridges were constructed from each shore to provide access to the water piers. Foundations



Sections through tubular cable anchor beam.

were prepared by removing shallow overburden with a clamshell and leveling the bedrock using divers and underwater blasting.

To prepare the supporting elements for the concrete piers, sheet piles were placed in the water, forming a sheet-pile cell. Water was not pumped out of these cells. Reason? It was impossible to get a watertight seal between the sheet piles and the channel bottom: there was no overburden over the bottom rock to give support to the sheets; nor could the sheet piles be driven into the bedrock. So, without attempting to remove water from the cell, the bedrock was merely cleaned off and concrete poured, with the sheet piles acting as forms. The concrete displaces the water and hardens, as well as, perhaps better than, that placed under dry conditions. The usual precautions were taken to avoid agitating the concrete. The concrete was transported with mixer trucks and placed with a concrete bucket and crane. The remainder of the pier concrete was formed with plywood and placed conventionally.

For the over-water steel erection, a 150-ton crawler crane with a 180-ft boom on a 60 x 150 ft barge was used; and a smaller crawler crane for the approaches. The girders were erected conventionally without falsework until they reached the main piers from each side. The pylons were then set in place and bolted down. Holes had been provided in the through plates at the top of the pylons to accommodate temporary erection guys. These supported the girders as they cantilevered out toward the cable anchors, 150 ft from each main pier. Once the cable anchors had been erected, the permanent cables were installed and the center girder sections dropped in place.

Pylon positions and cable-anchor elevations were adjusted to predetermined values, and cables equally stressed before adding deck concrete. The steel erector accomplished this by adjusting all three cables of a group simultaneously, using three center-hole hydraulic jacks working off a common manifold. The cable anchor sockets have a center hole to accommodate a threaded rod for tensioning purposes. The backstays were adjusted first to position the tops of the pylons, and the forestays were then adjusted to position the cable anchors. Deck slab sections varying from 50 to 100 ft in length were placed in a specified sequence to avoid overstressing the girders.

Structural steel had been given two shop coats of zinc paint in the shop. The contractor gave the steel a light brush sand blast in the field to remove dirt and stains before a final coat of zinc paint and a top coat of vinyl paint, pale blue in color, were applied. Finishing touches included a continuous low-level roadway lighting system and navigation, clearance, and obstruction lights for aircraft and marine traffic.

Economics

The bridge items came in at $1,960,000 or $52 per square foot. Representative prices include $145 per cubic yard for Class "A" concrete; 22 cents per pound for reinforcing steel; 46 cents per pound for structural steel, and $1.63 per pound for cables and fittings.

Credits

Credit is due to Dennis Nottingham, formerly with the Alaska Department of Highways, for the structural analysis and computer solutions. Barry Bergdoll was Project Engineer for the Department of Highways during construction. Everett McKellar was Superintendent and Al O'Shea was Field Engineer for Associated Engineers and Contractors, Inc. Substructure work was done by H. Flechsing & Co. of Missoula, Montana, and steel erection was by Don L. Cooney, Inc. of Tacoma, Washington.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
101 Park Avenue • New York, New York 10017

Address Correction Requested

BULK RATE
U.S. POSTAGE
PAID
NEW YORK, N.Y.
Permit No. 6662

Steel Cuts Cost in Middle-Income Housing

by Samuel Paul, AIA

Non-subsidized middle income rental housing is nearly extinct in New York City today. The high costs of construction and maintenance have made private investment in such buildings all but impossible to justify.

However, we found that this type of housing **can** be economically feasible — even in the New York metropolitan area. Georgetown House, an 11-story apartment building now under construction in the Gravesend section of Brooklyn, proves the point.

Two features in particular enabled owner/builder Edelman & Jacobs to construct this $3,200,000 structure as a profitable venture: (1) careful cost studies to determine the most economical structural design and (2) the inclusion of leased professional office space on the first floor to produce greater rental income.

Comparison of alternate structural framing systems led to the selection of a steel frame, open-web joist design that achieved significant savings over the reinforced concrete design originally contemplated. The floor-ceiling assembly for the steel system saved more than 15 percent of the cost of the concrete system, and the lightweight steel joist construction resulted in 30 percent less weight on the building's foundations. Weight of the steel framing, including joists, was only 12 lbs/sq ft.

The combining of professional office space and apartments is nothing "radically new"; we designed many such buildings in the 1950's and early 1960's. As rental trends and the apartment market in general changed, this planning approach became unpopular and, subsequently, dormant. In today's residential market, however, high rise may well be the only practical method left to private builders/developers for middle-income urban housing.

When completed in March, 1974, the building will contain 110 apartments including 10 efficiencies, 90 one-bedroom units, and 10 two-bedroom units. Rents will range from approximately $250 for the efficiencies up to $550 for the two-bedroom units. In addition, because of the cost saving methods used in their construction, the new apartments will have attractive amenities for tenants, including heating and air conditioning with individually operated incremental units.

Architect:
The Office of Samuel Paul, Architect
Forest Hills, N.Y.

Structural Engineer:
Abraham Hertzberg
New York, N.Y.

Owner/Builder:
Edelman and Jacobs
Brooklyn, N.Y.

Samuel Paul is founding partner of The Office of Samuel Paul, Architect, New York, N.Y.