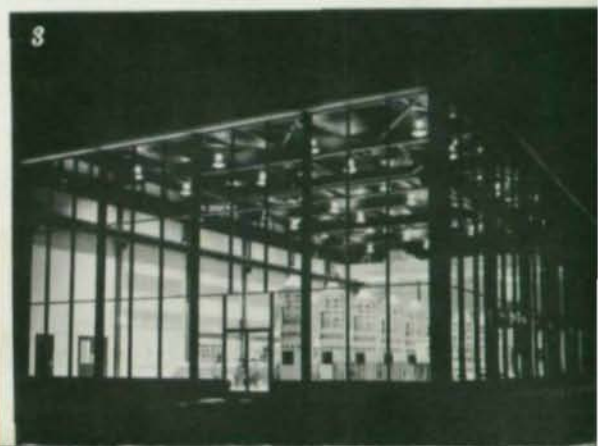
VOLUME XI / NUMBER 2 / SECOND QUARTER 1971

MODERN STEEL CONSTRUCTION



MODERN STEEL CONSTRUCTION

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1971 ARCHITECTURAL AWARDS OF EXCELLENCE COMPETITION

The American Institute of Steel Construction has announced the opening of its twelfth annual Architectural Awards of Excellence Program to recognize and encourage creative uses of structural steel. The 1971 Awards will salute outstanding aesthetic design in structural steel.

All registered architects practicing professionally in the United States are invited to enter steel-framed buildings of their design constructed anywhere in the 50 states and completed after January 1, 1970 and prior to September 1, 1971.

The structural frame of the building must be steel, although it is not a requirement that the steel be exposed and a part of the architectural expression. Buildings of all classifications are eligible, with equal emphasis given to all sizes and types in the judging. There is no limit to the number of entries by any individual or firm. Buildings named as previous AAE winners will not be eligible.

The Competition will be judged by a panel of five distinguished architects and engineers. The members of the 1971 National Jury of Awards are:

John P. Eberhard, AIA *Dean, School of Architecture and Environmental Design, State University of New York at Buffalo, Buffalo, New York*

James H. Finch, FAIA *Finch Alexander Barnes Rothschild & Paschal, Atlanta, Georgia*

Dahlen K. Ritchey, FAIA *Deeter Ritchey Sippel Architects, Pittsburgh, Pennsylvania*

Edward J. Teal, M.ASCE *Albert C. Martin and Associates, Los Angeles, California*

Max O. Urbahn, FAIA *President-elect AIA; Max O. Urbahn Associates, Inc., New York, New York*

Entries must be postmarked prior to September 4, 1971 and addressed to the Awards Committee, American Institute of Steel Construction, 101 Park Avenue, New York, New York 10017.

Structural Steel Helps Build Newark Airport Terminal Buildings

by Eugene J. Fasullo
and
Herbert Y. Chu

In this age of jet travel, airports must keep pace with the ever-increasing demand for additional facilities to handle rapidly expanding passenger volumes and larger aircraft such as the Boeing 747. The Redevelopment Program for Newark Airport, estimated at more than $200-million, is being carried out by the Port of New York Authority to meet this demand.

This project includes construction of runways, taxiways, and three terminal buildings. Each terminal comprises a main building connecting to three circular satellite buildings. In turn, these nine satellite buildings will accommodate a total of 83 DC-8 type aircraft

Mr. Fasullo is Chief Structural Engineer and Mr. Chu is Senior Structural Engineer with the Port of New York Authority.

Main Building — Departures and Arrival Level

loading positions. The terminals will be supported by initial parking facilities for 10,000 automobiles, a central heating and refrigeration plant, a fuel storage and underground distribution system, an airport facility building, many other small buildings, and an extensive highway network within the airport.

Two of the new terminals now under construction on the 427-acre site consist of a gently curved split-level main building approximately 900 ft long x 190 ft wide, with an average radius of 750 ft for Terminal No. 1 and 3,300 ft for Terminal No. 2. Three connectors, 29 ft wide with varying lengths ranging from 350 ft to 530 ft, lead to three two-level satellite buildings, 200 ft in diameter with a 50 ft diameter clerestory raised 9 ft above the roof at the center of the building.

Main Buildings

The three terminal buildings are designed to separate arriving and departing travelers. The upper (departures) level and the lower (arrivals) level consist of lightweight concrete slabs on metal forms supported by structural steel framing and are structurally independent from the adjacent elevated roadway system. Use of hyperbolic-paraboloid shells over the departures level provides unusual visual interest. Three larger h-p's located at nodule points form the major circulation area. Under the larger umbrella at the rear of the building are the concourse and mechanical levels. These levels are supported by structural steel framing.

Temperature and Structural Stability Considerations

Due to the temperature expansion and contraction, the entire main building is divided into three approximately equal units along the longitudinal direction, with expansion joints located at third points of all floor and roof levels. All floors and roofs supported by structural steel framing are self-stabilized for horizontal loads through rigid frame action in both longitudinal and transverse directions, except for the departures floor which is connected to the cast-in-place concrete columns supporting the hyperbolic-paraboloid umbrellas. The frame action is achieved by introducing moment connections between steel columns and girders at selected locations. At the rear of the building, the combined actions of concourse floor, mechanical floor and roof over mechanical level necessitates the use of multistory rigid frame design. As the rigid frame would deflect under live load and wind pressure at the rear face of the main building, the concourse floor framing is separated from the departures and arrivals floor framings by using sliding beam seats on column brackets and flexible plate hangers.

Curved Girder for Departures Floor

As the architectural design for the arrivals level does not permit any exposed steel columns supporting the departures floor system, long span construction was necessary. Beams 64 ft long with 22-ft long cantilevers were used to support the departure level sidewalk along the roadway. Due to the curvature in the building, these beams are oriented radially and supported by a five-span continuous curved girder consisting of a W36X280 with varying thickness of cover plates up to 3½ in.

The girder has equal spans of 54 ft and is supported on compact built-up W-shape steel columns 8 in. x 10 in. in cross section, which are encased in one of the four projections of a cruciform concrete column supporting the small hyperbolic-paraboloid umbrella. In order to prevent the interaction between the steel framing and the concrete column, a coat of bond-breaking compound was applied to the steel column and ¼-in. thick neoprene material was epoxied to the curved girder to allow for the axial shortening and angular rotations when the floor is loaded.

A special hinge detail consisting of two 1½-in. dia. dowels was used at the top of the steel column cap plate. As the final finished elevation of sidewalk at the curb line is designed to be 6 in. above the roadway under normal conditions, the tops of all cantilevers were set to different elevations in accordance with deflection calculations based on corresponding beam sizes and back span lengths. The control of beam tip elevations in the field was achieved by pre-cambering the 18 in. deep distribution channel connecting all cantilever tips and tilting the cantilevers about their moment connections at the girder web. Expansion joints in the curved girder were introduced at the third points of the building to match the location of other expansion joints. The joint detail has a bronze bearing plate with flat surface at the bottom to facilitate sliding caused by temperature changes and a curved top surface to allow the girder to rotate under live loading. The joint is designed to transmit a girder reaction of 350 kips.

Inter-Terminal Transportation System — (ITT)

An Inter-Terminal Transportation system is planned to connect the three terminal buidings with operations, such as remote parking areas, future hotels, and other services. Although the final choice for this automated system has not been made, steel columns and struts together with pile foundations spaced at approximately 60 ft on centers within the building limits were designed to support two lanes of traffic. The size of steel box-section columns varies from 30 in. x 30 in. to 30 in. x 42 in. The steel strut, 17 in. x 40 in. in size, is connected to the top of the column and is designed to accept four future stringers supporting ITT tracks at predetermined locations. This ITT structure is completely separated from the building framing.

Connections

The framing design for this building is rather complex. Beam sizes used vary from M6 rolled sections to 5-ft deep built-up girders, and column sizes vary from W10 to the ITT box sections. Simple span beams, continuous span beams or girders, horizontal trusses, posts supported on cantilever beams, rigid frames, etc., prevail throughout the entire building. Moment connections for beams to columns and beams to beams of different sizes have a great variation in magnitude of design moment and type of connections. To minimize the detailing required on the contract drawings, special types of connections were shown on the drawings. However, the exact design of these connections was left to the steel fabricator based on the forces given on the framing plans. These designs were then checked and approved during the shop drawing review.

Satellite Building

The satellite buildings are designed to hold air travelers before boarding the aircraft and to disperse air travelers upon their arrival through loading bridges connecting to the respective gates which are located along the periphery of the passenger floor. The framing for both passenger floor and roof basically consists of two-story rigid frames located on radial lines dividing the circle into 12-degree segments. The lightweight concrete floor and roof slabs are supported by parallel beams on chords between radial girders. All the periphery beams were designed for an assumed reaction from an anticipated future loading bridge. The frame action between girders and columns is achieved by moment connections. Since the satellite is only 200 ft dia., no expansion joints were required.

Steel framing of one of nine satellite buildings

Aerial view of terminal building and two satellites

Connector

The main function of the connector structure is to provide a link between the satellite building and the main building. The arcade structure comprises one-story rigid frames spaced at approximately 30 ft o.c. The framing for the passenger floor is connected to the bottom of the rigid frame which is supported by two independent reinforced concrete piers forming the shape of a hammer head supported on pile foundations. The space between the hammer heads under the passenger floor is designed to have conveyors installed to transport baggage between the operations level of the satellite building and the departures and arrivals levels of the main building. Along the two side strips of the passenger floor, provisions were made in the framing to have moving walkways installed in the future. Within the limits of these moving walkways, precast concrete planks were placed in lieu of the normal lightweight concrete slab. These planks can be removed if the airlines choose to install moving sidewalks. The lightweight concrete roof slab is supported by simple beams spanning between rigid frames. Expansion joints through the entire connector structure are introduced at approximately 210 ft on centers. The elevation of the connector will reveal two continuous bands of sand-blasted white cement precast concrete panels, one at roof level and one at passenger level. A glass wall is sandwiched between the two concrete bands. The space below the passenger floor is completely open.

Design Features

The main advantage of structural steel framing for all floors is the adaptability of this system to future alterations. In addition, the many special conditions, such as long spans, long cantilevers, floor openings, close deflection tolerance, shallow construction depth, as well as speed of construction could be met. Economic analysis proved its competitive position compared to other forms of construction.

A36 steel is used throughout the project. Uniform live load used for all levels is in conformance to the Building Code of Newark, New Jersey, 1964. In general, 120 psf was used in all of the public spaces.

Steel framing for Main Building

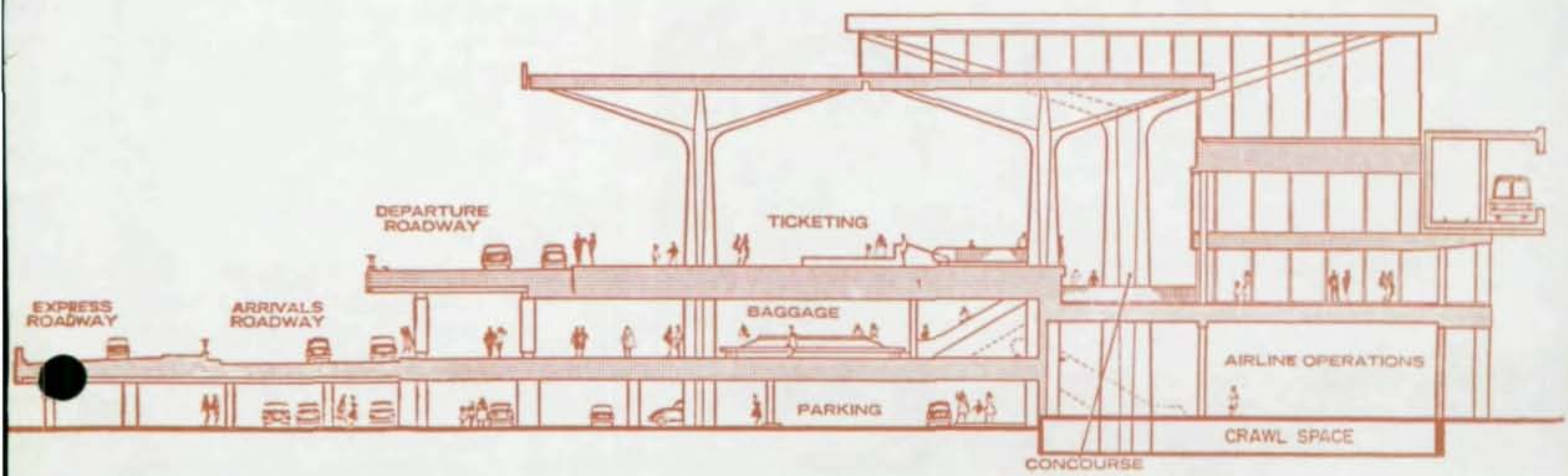
In addition to normal live loads, sidewalks on both arrivals and departures floors were designed to support a moving concentrated wheel load of 12,000 lbs. This accounts for a possible accident condition. On the concourse floor, a moving concentrated load of 2,000 lbs distributed over 2½-ft x 2½-ft area was used when stress in the beam exceeded that produced by the uniform live load. Along the sides of the connector floor, a total conveyor loading of 200 lbs per linear ft was assumed to be carried by the structural framing. Due to the open-air surroundings in the airport, the terminal buildings were designed to sustain a wind pressure of 20 psf on all exposed vertical surfaces, and 30 psf on all window assemblies. The ITT structure was designed on the basis of an assumed vehicle weight.

The entire framing system was designed on the basis of elastic theory, and non-composite action. However, shear studs at nominal spacing were added to those beams which have long spans and will support permanent window walls or masonry walls to create composite action for minimizing live load deflection. Camber based on dead load plus partial live load was specified on the framing plans for those beams which have deflection in excess of ½-in. In order to assure pedestrian comfort, all beams under the public assembly area were checked for undesirable vibration due to "heel impact." The comfort perceptibility is dependent upon the rigidity and natural frequency of the floor system which is measured in terms of deflection caused by a given load and ratio of span to depth of structure. For the type of anticipated traffic, the following criteria was applied:

$$\triangle < 0.005 \text{ in.}$$
and
$$L/d < 20$$

where $\triangle$ = deflection of a beam subjected to a concentrated load of 300 lbs at the center of the span; L = span of beam; and d = total depth of beam, including the thickness of concrete slab.

This design was conceived and developed by the authors and their staff of The Port of New York Authority Engineering Department. The entire project is under the direction of William P. Starr, Jr., Engineer of Design, and Martin S. Kapp, Chief Engineer. Fabricating and erecting the 12,000 tons of structural steel cost approximately $4-million.



Steel Serves Well for Tennis Club

Architect: Sasaki, Dawson, DeMay Associates, Inc.
Watertown, Massachusetts

Structural Engineer: LeMessurier Associates
Cambridge, Massachusetts

General Contractor: Charles A. Logue Building Co.
Needham, Massachusetts

Steel Fabricator: Inland Ryerson Construction Products Company
Chicago, Illinois

by Kenneth DeMay

Pre-engineered buildings eminently suit the problem of stringent budgets and building schedules.

In order to attain the necessary degree of economy, the new Hazel Hotchkiss Wightman Tennis Center, Weston, Mass., is basically a prefabricated steel building. The primary design goal was therefore to achieve within the limits of this system an aesthetically exciting building appropriate to a tennis club, without sacrificing the low cost inherent in this normally utilitarian construction.

The present program outline of the club is as follows: Indoors, there are

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Mr. DeMay of Sasaki, Dawson, DeMay Associates, Inc., was the Principal Architect in charge of the Hazel Hotchkiss Wightman Tennis Center.

four tennis courts, two squash courts, a 30 ft x 60 ft swimming pool, a lounge, lockers, showers, saunas, a snack bar, and the office and pro shop. Outside are ten tennis courts, a 45 ft x 75 ft swimming pool, parking for 100 cars, and a landscaped lawn area. The site consists of approximately 16 acres, with the south property line along the Massachusetts Turnpike. In order to isolate the outdoor tennis courts from traffic noise, a long narrow building was designed, and was located along the south line between the Turnpike and the outdoor courts. The outdoor pool, another source of noise, was also located along this line. The outdoor courts are kept to the north side of the site with a landscaped lawn forming a mall area between them and the buildings.

The building consists of three separate staggered sections. Two units are modified pre-engineered buildings, each housing two courts. The third section houses all other indoor facilities and is also a metal building, except directly over the indoor pool which is laminated wood beams on concrete block-bearing walls.

The building area covers a total of 44,115 sq ft. The metal building is made up entirely of standard "off the shelf" parts with several modifications for aesthetics. The two sections housing the four indoor courts cost approximately $10 per sq ft; the metal building alone was about $3.50 per sq ft, with the modified extras amounting to between $.25 and $.50 per sq ft. The third section which houses all other indoor facilities costs about $30 per sq ft; this unit includes the swimming pool, squash courts, carpeting, and lockers. The average cost of the total building was $16.80 per sq ft.

All spaces except tennis courts and pool are air-conditioned. Gas-fired packaged units heat the tennis courts. The heating and ventilating equipment is housed in a series of simple roof and wall extensions, utilizing the standard components of the building. These further contribute to the variegated silhouette of the building.

Without the benefit of "uncommon knowledge," designers had to depend on common sense.

Wooten's Third Law & Steel Column Design

by Jim Wooten
Chief Structural Design Engineer
AFCO Steel, Little Rock, Ark.

As you may know, Wooten's Third Law states, "The acquisition of uncommon knowledge inhibits the application of common sense." (Wooten's First and Second Laws are concerned with sex and, although they are much more interesting, are not germane to this discussion.) The "Absent-Minded Professor" is an illustration of the Third Law at work, but many other examples come to mind readily. For instance, the first use to which we put the discovery of nuclear fission was a bomb which had the potential of destroying us. The automobile is a marvelous application of uncommon knowledge, but its operators have displayed an amazing lack of common sense in its use and abuse. We have been so lavish with DDT and its fellow uncommon chlorinated hydrocarbons that some say we are now on the verge of wiping out all life except the insect which they were designed to control. Perhaps the best illustration of the Third Law is the computer, a machine which can absorb millions of bits of the most sophisticated uncommon knowledge and still remain abysmally stupid, becoming completely unhinged if one jot or tittle is misplaced in its program. Unfortunately, like the computer, we tend to become programmed rather than educated.

In the design of structural steel, plastic analysis has helped us to use common sense in designing continuous beams, but the Third Law creeps in when we attempt to analyze column stresses. A brief historical review (with carefully selected facts) may help to demonstrate this.

In the early days, designers did not have the benefit of the uncommon knowledge which extensive research has given us, and they had to depend on common sense. (However, some of the earliest structures were based on ultimate strength, which we think of as a "modern" technique.) Because they had no practical means of dealing with indeterminate structures, they made the structure determinate by assuming pinned ends and designing beams as elastic simple spans — thus using an important corollary of the Third Law which we shall discuss later. If the structure had to support lateral loads as a frame, they again made the frame determinate with assumed inflection points and developed only the computed frame moment at the beam-column connection, ignoring the moments caused by vertical loads on the beams. This apparent inconsistency seemed to work, and, under some conditions, it was not overly conservative. (For a detailed discussion, see "Wind Connections with Simple Framing" by Robert O. Disque, **AISC Engineering Journal,** Vol. 1, No. 3, July 1964.)

For column design, the Fruit Salad (F.S.) factor was concocted, wherein the proportion of the column strength used by the axial load at one allowable stress (apples) is added to that used by bending about one axis at a second allowable stress (oranges) and to that used by bending about the other axis at a third allowable stress (pears). Mathematically, it is difficult to add apples to oranges and pears, but the idea appeals to our common sense as well as to our taste.

Many structures were built using these simple assumptions and many are still giving dependable service. The designers probably were pragmatists who decided that they would never know what the actual stresses in the members were, and they didn't care as long as the structure did not collapse and was not too uneconomical. Those fortunate few who are not rich, handsome or charming, yet score well with the opposite sex, never ask questions— they just enjoy their success. However, it might profit the rest of us to search for the answer.

Continuing our history, things rocked along well until the early thirties, when Hardy Cross published his papers on moment distribution. This brilliant, common sense idea did much to dispel the fear of indeterminate structures and helped speed the burgeoning development of monolithic concrete design. Steel design did not change much, but simple beams were not very glamorous and the designer felt a little guilty because he had no elastic moments to distribute.

The development of plastic design in the fifties was a common sense approach which explained the true action of continuous beams and single-story frames under stress and should have quelled the elastic stress syndrome which began to infect steel design; however, its use in columns was delayed for so long that common sense was inundated by the Computer Age.

The computer's rapid solution of stupendous slope deflection equations and massive stiffness matrices renders obsolete the necessity of rationalizing and simplifying problems — or even of understanding them. No one need feel guilty of using simple solutions when the computer can make them extremely complicated. Anyone can plug a false assumption into an incorrect formula and, in a flash, arrive at a ridiculous answer, inaccurate to ten decimal places. (It has never been clear to how many places an inaccurate answer must be carried to make it accurate.) The computer experts warned us at the beginning "garbage in — garbage out," but in our rush to compute and add elastic stresses to arrive at a numerical answer, we have lost our perspective and the ability to determine if the structure really acts in this manner.

We have modified the F.S. factor by applying a *K* coefficient to the column length, and an amplification and reduction factor to the bending ratio — rational improvements which help us understand the parts of the factor better — but we overlook the fact that this is basically the same old fruit salad with maybe a little mayonnaise added for smoothness. To insist that a column design check within 1 percent or 2 percent of such a factor is absurd.

For that matter, to think that we can add computed elastic stresses at any point in a member and arrive at a dependable answer is absurd. The original heat and the rolling and subsequent cooling of the particular shape introduce certain residual stresses. Any fabrication process — cutting, coping, punching, cambering, or welding — impose further residual stresses and if the member does not fit perfectly in the field, only God knows what happens to the poor thing before it is finally laid to rest. Since we don't even know what state of stress the member is in before any loads are applied, it is hardly logical to think that we know more about it after we apply certain assumed loads which are probably 50 percent to 100 percent wrong. This is why it is irrational to base allowable stresses on a certain yield stress at which the member is assumed to fail. We must "fail" the steel elastically to cut it, punch it, or perform any act of fabrication on it. A bent plate connection has been "failed" before any load is applied. Yet plastic design research has shown that these residual stresses and elastic "failures" in the member do not interfere with the attainment of its theoretical ultimate strength.

Thus we must confess that, with all our uncommon knowledge, we cannot compute the actual stress at any point in member. Having conceded that, we can see it never was important anyway. We only used it to arrive at an idea of the consequent *strain* in a structure — the deformation, deflection, or possible cracking and tearing — which is what we are really interested in. Hooke's Law, upon which elastic analysis is based, is never valid throughout the structure; however, it can be used to approximate the likely deformation under load, and if our structure meets the limitations of strain criteria, then we don't care what the stress at any point might be.

This leads us to the previously mentioned corollary of the Third Law; that the material in a structure, unencumbered by any uncommon knowledge whatsoever, is smarter than the engineer who is not so blessed. It follows that all the engineer has to do is to size the members to accommodate the total moments in a structure, making any "rational" assumption as to the distribution of these moments and considering the consequent strains therefrom. The material will then conform to this assumption. Obviously the best assumption to use for a particular case is that which produces the lightest structure for a given allowable strain.

Before proceeding further, it is necessary to add the following qualifications to the above statements:

1. The material is assumed to be structural grade carbon steel which has the necessary ductility to conform to almost any stupid assumption.

2. The loads are assumed to be the customary static loads found in buildings with the possibility of few load reversals.

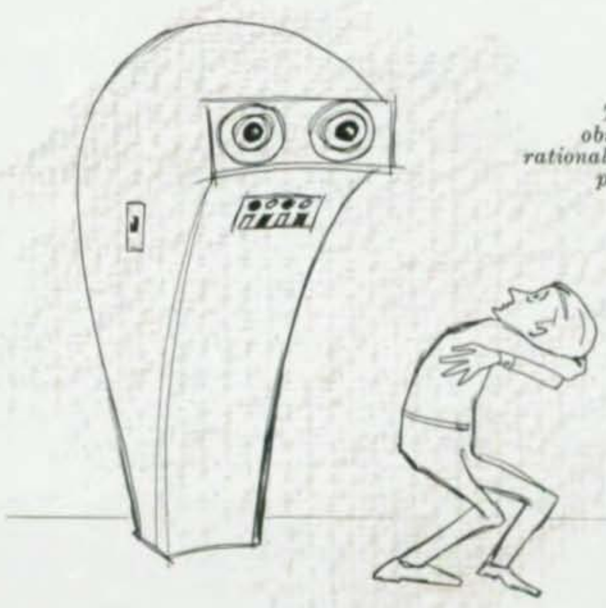
3. The members must be properly proportioned and braced to allow rotation without local or lateral buckling as specified in the rules for plastic design. (This is not always as easy as the textbooks imply; however, with some ingenuity, it can be accomplished with little overall expense).

4. Except for low structures with a few stories, sidesway of the frame must be prevented by a positive bracing system such as diagonal braces, shear walls, or anchorage to a braced structure. (This is probably not necessary, but this qualification does eliminate a lot of worry and argument. The world is not ready for the full Wooten treatment — even Einstein had to limit his work to a "Special" Theory of Relativity until he later produced the "General" Theory.)

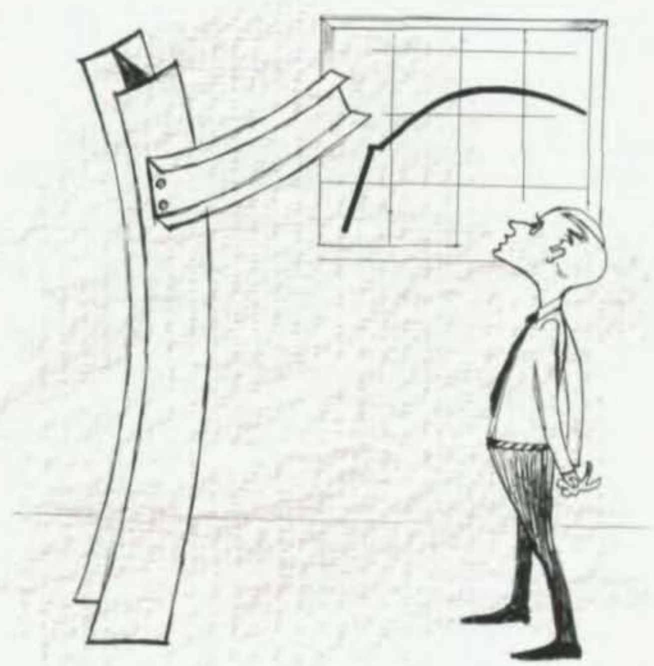
5. It is assumed that the members are of sufficient stiffness so that deflection or deformation is not a problem. (If this is not true, then an investigation of stresses is pointless and the members would be designed for stiffness only.)

The above qualifications are not unduly restrictive since the overwhelming majority of steel buildings are designed in line with these provisions for the sake of integrity or economy.

The simple beam moment diagram for vertical loads and the portal frame analysis for lateral loads are simple computations that identify the total moments which the structure must support. Knowing these, the designer may make any "rational" assumption he



The computer renders obsolete the necessity of rationalizing and simplifying problems — or even of understanding them.



The material in a structure, unencumbered by "uncommon knowledge" is smarter than the engineer who designs it (Wooten's Corollary).

wishes as to the distribution of these moments and the material will conform to this assumption. Considering vertical loads only, he may assume pinned ends and design the beam for maximum moment at midspan and the column for axial loads only if the structure is braced laterally. Or he may assign any portion of the total moment to end fixity, reducing the moment at midspan correspondingly, provided that the end connection and either the adjoining beam, if present, or the column, or both acting together, be strong enough to develop this moment. This can be carried to absurdity by assigning the total moment to the ends and placing a pin at midspan, and the structure will be stable although probably not economical. The same procedure can be used to analyze moments due to lateral loads. It should be remembered that the strain of the members will vary as the material conforms to the various assumptions made.

There should be nothing very upsetting about Wooten's Corollary, since we use it every day without admitting it. Our standard bolted shear connections, which are assumed to act as pins, must yield and deform considerably before rotating or distributing the load equally to all bolts as we also assume. We design deep trusses in industrial buildings as simple spans with pinned ends, then connect the ends to the columns to develop the portal frame moments from lateral loads, ignoring the gravity moments induced in the columns. We connect a deep girder to the web of a long slender column by means of a long bolted connection to develop "shear" only and have no qualms about designing the columns for axial load only, yet the connection will develop the full elastic moment of the column and produce an F. S. factor of 1.5 to 2.0. Should a beam, intended to have a "shear" connection only welded to its web, by mistake have its flanges welded to the columns, it is highly unlikely that the designer would insist on the flanges being cut loose.

None of these examples conform to the design assumptions, yet they are successful because a plastic hinge develops in the beam, the connection, or the column, and the connection behaves as a pin from that point onward without stressing the member.

We ignore the moment at the connection when the girder is much stiffer than the column because so little elastic moment can be developed by the slender column (of course, it can't resist much either); however, when the stiffness of the beam and column are comparable and the connection fairly rigid, as is the case in multi-story frames, we ignore plasticity and carry the elastic moment into the column — yet the principle is the same as in the examples listed.

It is not necessary to combine moment induced at a connection with other moments or loads in a member unless an adjacent beam or column is dependent upon this moment for its stability, because the end of the member, which is braced, will yield and rotate, thus protecting the remainder of the member, which is not braced, from dangerous overstress.

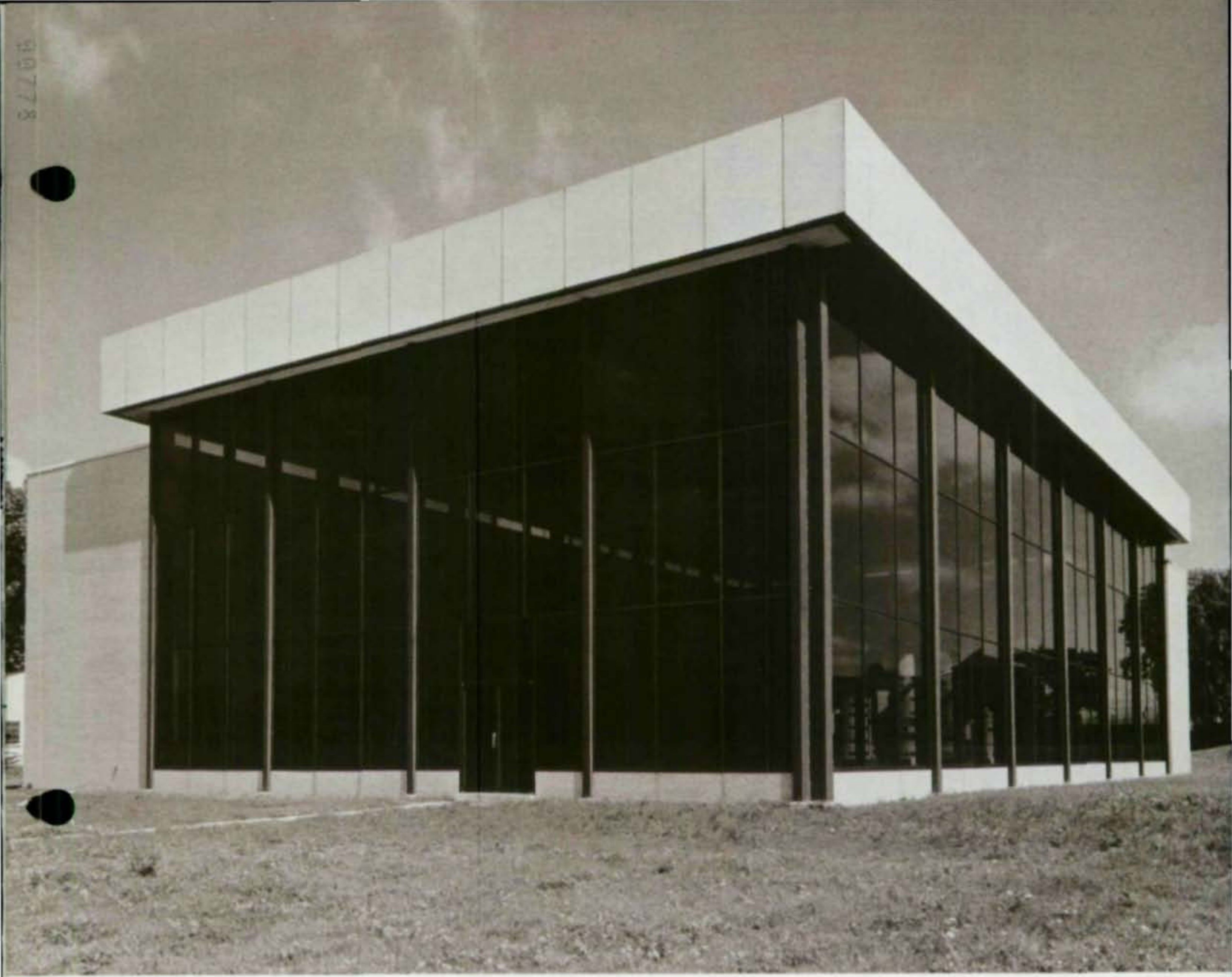
Once the Third Law and Corollary have been accepted, a design will be limited only by the designer's ingenuity and not by a computer's storage capacity. We can forget the so-called "semi-rigid" connection, which attempts to control the strain by sizing the material on the basis of theoretical elastic stresses when the state of stress in the material is never known before, during, or after loading. We can forget moments induced in columns by checkerboard loading, because continuous beams, designed for maximum loads, will develop the unbalanced moments from reduced loading without help from the columns. Depending on the relative lengths of each, we can use the moment capacity of a column to assist the beams or ignore the elastic moment if it is of no economic advantage. Summarily,

1. The acquisition of uncommon knowledge inhibits the application of common sense (Wooten's Third Law).

2. Structural Steel, having no uncommon knowledge, is smarter than the engineer who designs it (Wooten's Corollary).

3. *Elastic stress* in a braced steel member is usually meaningless, which is fortunate because it is impossible to compute. If the consequent *strain* of a structure is acceptable, the only stress criterion is that the *total* structure be proportioned to support the *total* simple moments induced by the loading. Consequently, "semi-rigid" connections and "checkerboard" stresses are wasteful placebos for the theoretician, and do not contribute to the physical health of the structure.

4. Wooten's First and Second Laws are more interesting.



Steel Modernizes Pumping Station

Architect-Engineer: Alden E. Stilson & Associates
Columbus, Ohio

General Contractor: C. & C. Construction, Inc.
Fort Wayne, Indiana

The Scioto River Pumping Station replaces the old station which was built in 1909. The new station provides water to approximately one-half the population of the City of Columbus, Ohio. The new station features the most up-to-date pumping equipment (total capacity 120 MGD) and sophisticated controls. The building structure was designed to the same high standards of the equipment and controls.

Functional Requirements

Requirements dictated a column-free pump room, approximately 60 ft x 110 ft x 30 ft high with an overhead traveling crane 22 ft above the floor, plus space for electrical equipment, workshop, storage, office, lavatory, and stairs.

Architectural Solution

A scheme was developed to satisfy the design requirements as follows:

- Structural steel boxed columns at 18 ft centers were selected to support the crane runways, as well as the space frame roof structure.
- The south and east walls (nearest State Route 33) were designed to receive fixed tinted glass subdivided by fixed steel tube mullions, horizontally and vertically.
- Masonry walls on the west and north sides were used to enclose areas containing electrical equipment, workshop area, storage, office, toilet room, and stairs.
- An overhanging eave and cast stone fascia were used around the large pump room. This eave and fascia are supported structurally by steel framing over columns.
- The structural steel is for the most part exposed. The exterior glazed brick was also used in the interior of the pump room to achieve architectural continuity.

Structural Solution

Structural steel rigid framing, 60-ft long x 30-ft high, spaced at 18 ft centers, was selected to best meet the functional requirements and aesthetics of the architectural design. The columns were utilized to support the crane runway which, in turn, provided stiffness to the columns.

Three-dimensional deflections along the plane of the walls were checked and held to a minimum to avoid cracking in the glass walls.

Welded connections were predominantly adapted to provide a clean finish.

Conventional cross bracing was provided in the plane of the roof to resist wind loads and crane loads.

Lighting

Interior lighting was designed to create a dramatic nighttime view of pumps and piping through the south and east glass walls.

Future Planning

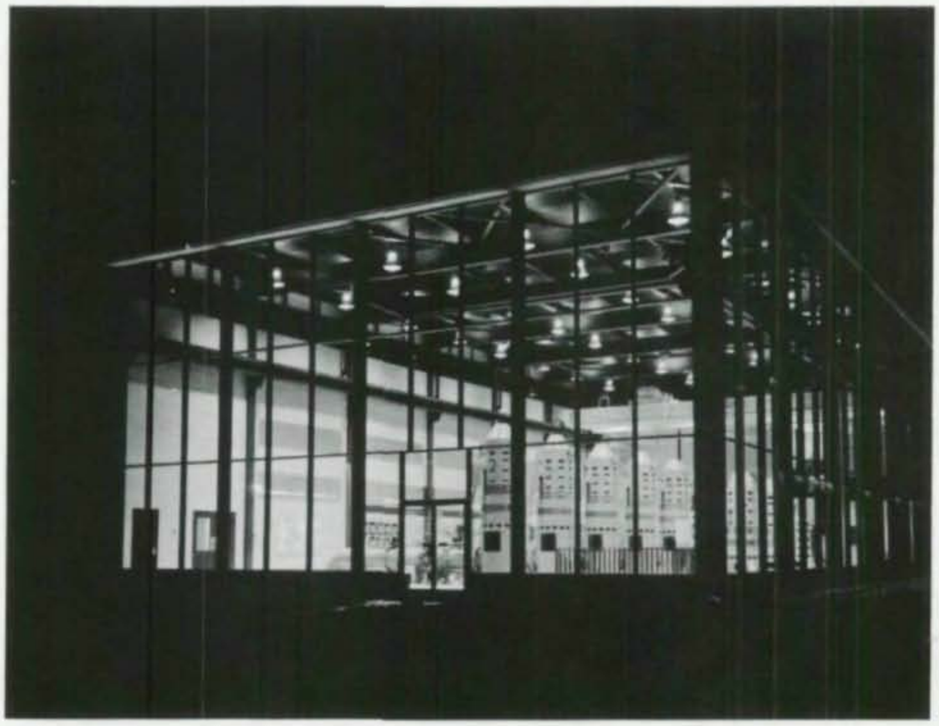
This building is the first phase of a complete treatment plant facility. The remainder of the plant has been designed; however, construction has not been authorized.

Cost

The total construction cost of the pump station was approximately $4,300,000.

Siting Problems

Siting of the Pump Station and the future Filter Building and Chemical Building is such that the existing facilities can be used for continuous operation until the new facilities are ready to be used. No shutdowns are permitted.



Recording without Red Tape

The corporate headquarters of GRT — a leader in the recorded tape industry — is the most recent addition to the company's general master plan. Located on a 6½-acre site in Sunnyvale, California, the new building has been carefully related to the company's main plant across the street. Primarily an office building, it also houses some special purpose sound studios and engineering spaces.

The building has been designed as a series of flexible office wings, arranged in parallel between an executive wing in front and a service wing in the rear, and separated by lanscaped courts. The plan is organized about the two circulation corridors; a major two-story spine and a secondary service corridor. Their importance is emphasized by strong vertical design expression with continuous clerestory windows.

Flexibilty Tested

A primary requirement was to provide flexibility and expansion capability to match the company's rapid growth and evolution. This capability has already been tested. While in working drawings the fourth office wing was added, and numerous layout changes have been made during construction. There is capacity for one additional office wing as well as extension of both the executive and service wings.

The office wings are designed on a 5-ft module. The specially designed 5-ft square ceiling unit contains the necessary lighting, heat and air-conditioning capability, as well as provision for partition attachment. Carpeting extends throughout under the partitions, and underfloor electrical and communications ducts are also provided for maximum flexibility.

Architect:
Brown/McCurdy/Nerrie
San Francisco, Calif.

Structural Engineer:
Hirsch & Gray
San Francisco, Calif.

General Contractor:
Johnson & Mape Construction Co.
Menlo Park, Calif.

Steel Fabricator:
Pittsburgh-Des Moines Steel Co.
Pittsburgh, Pa.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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Recording without Red Tape (continued)

Structural System

A shop-welded, field-bolted, steel rigid-frame system on spread footings with steel bar joists and deck was chosen. Reasons for selection included flexibility of space arrangement, economy, speed of erection, non-combustibility, and ease of maintenance.

The exterior skin of the building consists of conventional industrial metal siding panels rolled of weathering steel. The structural steel frame, where exposed, is also of weathering steel. The window wall is made up of weathering steel structural sections, neoprene gaskets, and bronze glass. In the short time the building has been erected, it has turned a pleasant warm red-brown color. Light and heat reflective glass is used where required for sun control and a reflecting plastic film was applied to the skylight at the main entry.