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MODERN STEEL CONSTRUCTION



1 Making a Bank Talk 8-9

2 The Aging Building Too Valuable to Replace 3

3 Stress and Strength Matched in Bridge Girders 12-13





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CONTENTS

The Aging Building Too Valuable To Replace	3
Delta Girders Offer Advantages for Long Spans	4-5
New Member Joins Structural Family	6-7
Making a Bank Talk	8-9
Questions and Answers on the AISC Spec	10
Steel School for the Handicapped	11
Stress and Strength Matched in Bridge Girders	12-13
Earthquake Teaches Structural Lessons	14-15
List of AISC Publications	16

EDITORIAL

Since the introduction of the 1961 AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings in mid-January, over 7000 architects and engineers have attended the four announcement conferences in Chicago, New York City, San Francisco and Dallas, and over 35 meetings in cities across the nation. These meetings have explained the "why" of the new Specification and the significant changes that have been made.

The Institute will follow up these one-day sessions with an extensive series of educational lectures to tell the "how" of using the Spec's more refined design provisions. These lectures will be arranged as afternoon or evening meetings once a week extending over several weeks. Engineers are invited to contact the nearest AISC regional office in the cities listed to the left for dates and plans for the proposed lectures. Answers to inquiries on the Spec itself may also be obtained from these offices.

The new AISC Spec has already been accepted in a number of cities where buildings using these new design provisions have been approved on an individual basis.

The Aging Building Too Valuable to Replace

The famous 700-ft Metropolitan Life Insurance Company tower building on New York's Madison Square Park was over 50 years old — and looked it. Marble facings and columns were beginning to deteriorate; interior equipment needed modernizing; and the general style of the tower gave scant sign of belonging to the same up-to-date, growing organization that had recently erected the clean-cut limestone structures that complete the company's four-block-square headquarters.

Still, the tower was priceless. For over half a century its appearance on all Metropolitan literature and letterheads had subtly advanced the company's image: a tower of strength. Because the building's steel skeleton is in fact as strong as the year it was

built, Metropolitan has been able to have its tower and modernize, too.

Two large sections of facing — marble alternated with limestone — are being replaced with two plain bands of four-inch-thick limestone backed with brick from the first through the fifth and from the twenty-ninth through the thirty-sixth floors. The load-bearing marble columns of the colonnade are being replaced with stone-faced steel columns. All carved ornamentation and horizontal emphases such as ornate cornices and heavy striped masonry quoins are being replaced with simple moldings and unadorned limestone. The trussed steel corner columns were cleaned and painted while exposed (there had been no damage to them over the years), and all marble facing which remains will be cleaned to blend with the new limestone.

Inside, modernization includes new plumbing and operatorless elevators, air conditioning, acoustical ceilings, and pivot-type windows that can be washed from inside the building. Renovation is scheduled for completion in February, 1963.

First finished in 1909, the Italian Renaissance style structure was then the tallest building in the world. (Today it is ninth tallest in the city and tenth tallest in the world.) The American Institute of Architects cited it as "the most meritorious work of 1909" for its solution of one of the most difficult problems of the day—skyscraper design. The tower represents one of the first uses of the steel columns that were to make possible the future's great leap upward.

So much more is known today about allowable stresses that the same building could now be erected using approximately 65 per cent as much steel.

New York history buffs are as pleased as the company's management that the sturdy old structure could be saved, for it is not only a trademark but a landmark. In 1957, the New York Community Trust designated the tower as a part of New York eminently worthy of preservation. To date, only eighty-some structures and sites have received such awards of historical significance. The tower was originally designed by N. LeBrun & Sons. Fabrication and erection was done by the American Bridge Division of United States Steel Corporation.

Architect for the renovation is Lloyd Morgan; general contractor, Starrett Bros. & Eken; structural engineers, Purdy & Henderson, the same firm that designed the original frame. All are from New York City.



Pre-modernization photo of Metropolitan Life Insurance Company's NYC headquarters shows, left to right: limestone building completed in 1941, an annex built about 1920; the 1909 tower; and the original structure at Madison Avenue and 23rd St., completed in 1906.



In February, 1963, Metropolitan's headquarters will look like this. Going: gold leaf on the roof and marble gingerbread. Staying: sound steel skeleton, famous four-sided, chiming clock, and a priceless trademark. Building in foreground is south wing completed in 1960.

DELTA GIRDERS OFFER



The 240-ft main span of the Parker Bridge near Yakima, Washington, comprises two 60-ft double delta cantilevers carrying a 120-ft suspended span.



The lateral stability of the rugged delta girder makes it easier to erect than conventional plate girders and precast concrete girders.

Delta girders for the Parker Bridge in Yakima, Wash.—a 240-ft main span flanked by 75-ft side spans — display the many advantages of these highly stable sections. Structurally, the delta girder's great torsional and lateral rigidity minimizes lateral bracing requirements and facilitates handling and erection. Architecturally, the girder presents clean, smooth lines, uncluttered by the usual stiffeners seen on plate girders.

Moreover, the delta girders are economical. The 400-ft-long bridge was bid at \$158,000 — \$30,000 less than an alternate steel truss design, and at a 16 per cent savings for the taxpayers of Yakima, Washington.

A delta girder comprises a basic plate girder section — web, top and bottom flanges — with the addition of two continuous diagonal plates welded to the compression flange and web to form a two-celled continuous triangular, or delta-shaped, box. At the Parker Bridge, the two 75-ft side spans — with their 60-ft cantilevers making a total fabricated length of 135 ft — are double deltas — i.e., both top and bottom flanges have the continuous diagonal plates. The 120-ft suspended-span girder, centered in the 240-ft span between the two main piers, is a conventional delta girder, with the diagonal plates occurring only at the top flange.

In elevation, the two 135-ft-long end sections taper with straight soffit lines from a maximum depth of 8 ft-5 in. at the main piers to 4 ft-8 in. at the abutments and 5 ft-9 in. at the cantilever ends. The suspended span is a nearly prismatic section, with 6 ft-2 in. maximum depth at mid-span. The three support points provided by the web and the two diagonal coverplates make possible the wide compression flanges for delta girders. The Parker Bridge has 36-in.-wide top flanges, with thickness varying from $\frac{3}{8}$ -in. to 2 $\frac{1}{4}$ in. The wide top flange makes an excellent surface for composite action. Studs and numerous

ADVANTAGES FOR LONG SPANS

small weld points transfer shear between the roadway slab and the top flange.

No top or bottom lateral bracing is necessary, since the wide flanges, cross frames and roadway slab combine to provide the necessary lateral strength.

The delta girder also facilitates fabrication. Unlike the vertical stiffeners that they replace, the diagonal plates, being longitudinal members, can be machine-welded. Still another advantage resulting from elimination of vertical stiffeners is the easier job of sand-blasting the girder surface and painting it.

Erection of the Parker Bridge — in particular the suspended span — displayed still more advantages of these rugged girders. First the 135-ft end-span and cantilever sections were placed. The end spans extended two feet into the massive concrete abutments, which anchored them against uplift but provided for horizontal movement.

Then a truck crane, braced and leveled at the river's edge, with a bridle sling attached at mid-span, picked up a 21-ton suspended span girder and swung it out over the river. A steel cable attached to the girder's far end passed through a block mounted atop a ginpole on the far-side supporting girder. It terminated at a winch mounted on a truck braced against the abutment. After the crane's bridle sling had been shifted to the near end of the girder, the ginpole cable and the crane lifted the girder into place on the bearing seats at the cantilever ends.

An equally long conventional plate girder handled this way would threaten to buckle. A 120-ft prestressed concrete girder picked up at mid-span would crack from negative bending unless it was reinforced for erection stresses.

Five load tests conducted last year at the University of Washington on a single delta girder established consult-

ing engineer Homer M. Hadley's faith in the delta girder. The test girder measured 36-in. deep at ends; top flange plate, 24 x 1/2 in., bottom flange 16 x 1 in.; web plates and top flange diagonal plates, 3/4-in. thick. The unsupported web height was 27 inches. Bearing stiffeners were welded at end supports for all of the tests.

The first test, under third-point loading on a 60-ft span, resulted in tension flange yielding with sudden, though slight, increase in deflection recorded as the girder sections at the load points became plastic hinges. Yet despite the vertical deflection, measurements from parallel piano wires showed no lateral deflection of the web.

The next two tests produced higher shear stresses. Two 20-ft lengths cut from the original girder were loaded with single mid-span loads after stiffeners had been welded at the loading points. One girder sustained a unit web shear stress of 17,950 psi; the other, 18,870 psi. In both girders the triangular delta section remained essentially unaffected.

Test Four, conducted on the 18-ft center section of the original girder, showed the performance of the girder

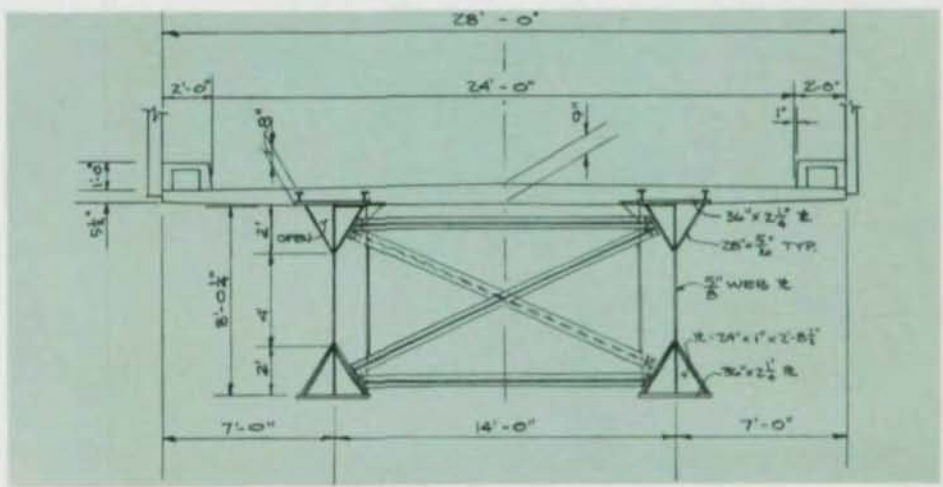
without stiffeners under the applied load. Before the beam reached ultimate load, the web, beneath the center line load and delta (an area about four feet long and one foot deep), bowed outward. This occurred at a shearing stress of 14,030 psi.

The last test, on the same girder, showed the torsional resistance of the delta girder. This torsional resistance was so great, in fact, that it could only be attributed to previously uncounted lateral restraint provided partially by the testing head itself.

The laboratory tests, however, were generally more severe than actual field conditions, where the roadway slab and diaphragms brace the girders.

Homer Hadley, Seattle consulting engineer, designed the Parker Bridge. Hans Skov Co. of Yakima was general contractor. Don L. Cooney, Inc. of Tacoma, Wash., was the steel erector. United Concrete Pipe Company, Auburn, Wash., fabricated the structural steel.

Aided by the university's civil engineering department, Mr. Hadley conducted the load tests at the University of Washington. The Pacific Northwest Steel Fabricators and the AISC helped sponsor the testing program.



Double delta girders of the Parker Bridge reach greatest depth at main piers. 3/4-in. web plate was only five feet long and was centered over bearing; adjoining webs were 3/4-in.

New Member Joins Structural

By Robert O. Disque

Assistant Chief Engineer, AISC

Nature has long understood the inherent efficiency of the tube. Reeds, flower stems and the limbs of men and animals are all tubular — and for good reason. It has only been recently, however, that round and square tubes have been used significantly in building construction.

The compressive strength of a structural member depends primarily on the geometry of its cross-section. In fact, Leonhard Euler's classic column formula shows that, in the range of elastic buckling, the moment of inertia is the only factor determining the load-carrying capacity of a steel column. The round and square tubes represent the optimum in the ratio of moment of inertia to weight.

Figure 1 shows the comparison of weights of structural members used as long columns. Although it is not possible to pick exactly equivalent cross-sections, it is apparent that weight savings of 36 per cent are possible with square tubes over angles and rolled columns; while weight savings of up to 53 per cent can be effected by using round tubes. Relative costs are, of course,

affected by material costs. Even with higher unit costs, tubular columns often show a net saving.

Prior to the advent of welding, connection difficulties inhibited the use of tubes as structural members. It was just not possible to buck-up a riveting gun or turn a nut from inside a tube. Welding, however, has alleviated the connection problem. With a square or rectangular tube it is just as simple to weld a clip angle or connection plate to a tube as it is to a rolled section.

With round tubes there are several methods of making connections, depending on strength requirements. Machines have been developed that will automatically cut a round tube to fit the surface of another tube at any angle and of any size. If greater strength is required, a gusset plate can be attached to the surface of a tube and connecting tube welded to the plate. Spheres, made by welding together two hemispheres pressed from plate, are used as a kind of three dimensional gusset plate. A number of square ended round tubes with their axes in line with the center of the sphere will fit perfectly on the sphere's surface by merely cutting off their ends square.

This spherical "gusset plate" system



Square steel pipes and rolled sections were used as the frame for the Burger Boy Food-O-Rama store in Columbus, Ohio.

offered a good solution to the complicated geometry involved in the new stadium to be built in Pittsburgh for the Steelers and Pirates. This project, designed by Deeter and Ritchey, Michael Baker, Jr., and Osborn Engineering will have a capacity of 55,000. A three-dimensional tubular truss system spans between twelve hollow concrete piers located on a horseshoe plan. In cross-section the truss system resembles the letter "C" — the top portion cantilevering over the grandstand to support the roof. The space frame is made from round pipe up to eight-inch diameter and hollow steel spheres 21 inches in diameter for "gusset plates."

Until recently only a few mills made tubing especially for structural purposes. The sizes and wall thicknesses of pipe and round tubing offered for pressure purposes are, in many instances, not the optimum for structural applications. Also, the steels and testing procedures are intended to prove the material for internal application and the steel grades are not equivalent to the grades used for structures.

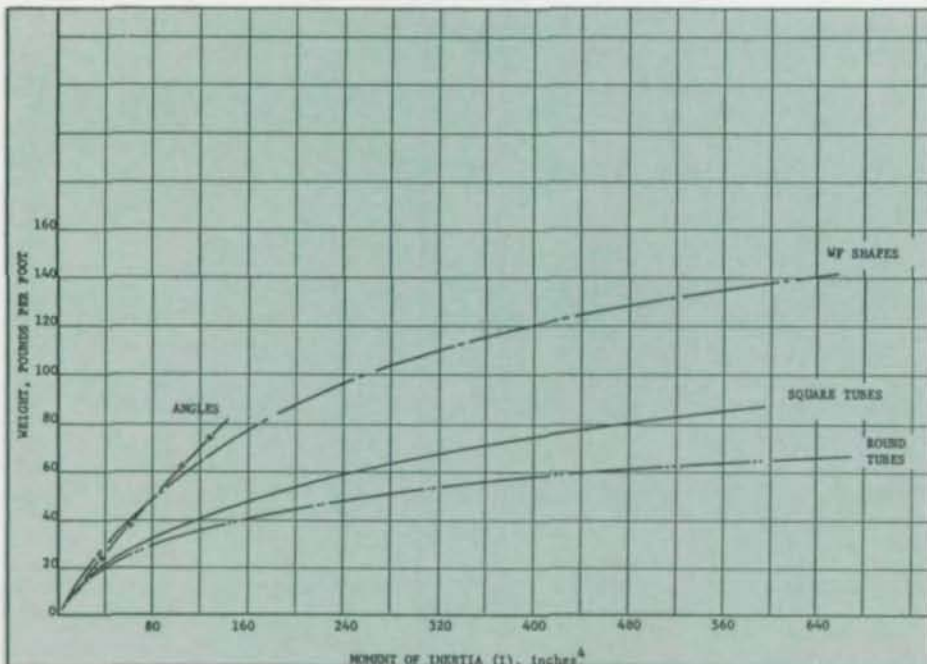
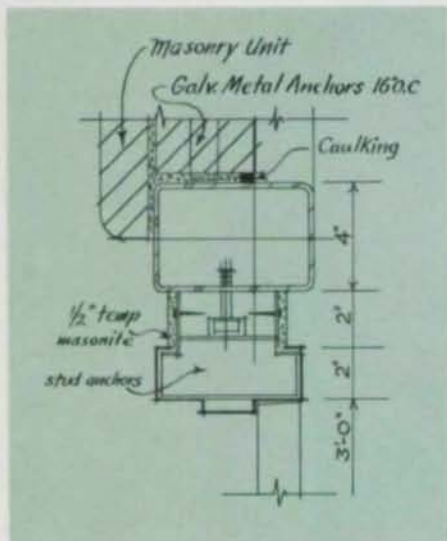


Fig. 1—Chart shows comparison of weights of structural members used as long columns.

Family



Jamb detail for Food-O-Rama shows how the three components of steel, glass and glazed masonry were joined together.

Round, square and rectangular tubular sections are now available from several sources, engineered entirely for structural application. ASTM A7 and ASTM A36 steels are used and the dimensions, properties, tolerances and other characteristics are related to the building field. Beam and column load tables, based on AISC specifications, are available from the producers of round, square and rectangular tubes.



The branch office building for the Union National Bank, Youngstown, Ohio, is a circular structure framed with tubular columns. The thin steel folded plate roof spans 50 ft with no need for interior supports.

Architects and engineers in Europe have made widespread use of the tubular section. Multi-story office and apartment buildings as well as industrial structures and transmission towers have been built with tubes. Weight savings of as much as 40 to 50 per cent have been achieved. It is interesting to note that one of the reasons for this was that European practice allowed greater efficiency in column design than permitted in the United States.

Prior to the adoption of the new 1961 AISC Specification, the column formula used in the United States followed a parabola in which the allowable compressive strength at zero l/r was appreciably less than the tensile strength of the material. The new AISC Specification will now permit greater efficiencies in the use of tubular columns.

The aesthetic qualities of tubular members are striking. Architects may use exposed round, square or rectangu-

lar columns painted any color of the rainbow to harmonize with the masonry or curtain wall. A good example is the Milwaukee Headquarters of the Wisconsin Division of the American Automobile Association. This building, designed by Rashe, Schroeder, Spransy & Associates of Milwaukee, is a complete steel frame including steel joists for the floor and roof framing.

Architect P. Arthur D'Orazio of Youngstown, Ohio, dramatically expressed the slender tubular columns in his design of a branch office for the Union National Bank in Youngstown.

A third example is the Burger Boy Food-O-Rama, Columbus, Ohio, in which architects Eiselt & Eiselt combine steel tubing, glass and glazed masonry units to achieve a smooth, clean, textured effect. The neat, attractive design permits quick erection and easy maintenance as well as establishing a "trademark" for the organization. Eight stores are planned for the Columbus area.

It is predicted that architects and engineers in the future will make greater use of this new member of the structural steel family.



The use of exposed steel tubes for the Wisconsin Division of the American Automobile Association formed an interesting facade, and, according to the architects, was very economical.



Two-story plan for the Community National Bank, Bakersfield, Calif., was dictated by the plot size and the need for on-site parking.

MAKING A BANK TALK

There is only one limit to buildings a designer can create with structural steel: his own imagination. Steel's strength wastes no space; and like a magic wand, it confers on the designer the power to make a building do and say whatever he bids.

With the circular new Community National Bank of Kern County, the message put across by David R. Harkness, AIA, Architect and Associates, was an inviting one: hospitable concern for the customers' every need. Paved parking

facilities, visible and easily accessible for Chester Avenue traffic, provide cover from sun and shower. In addition to the attractive interior facilities — including a board room which may be used by civic groups — are exterior windows for drive-up and walk-up banking for added customer convenience.

Those bound for the second-floor loan offices may choose either an elevator or stroll up a gently inclined, curved and covered exterior ramp that has a group of trees and shrubs to its right

Steel Construction, September, 1961). Still, its massive, jutting vault area, finished with mosaic tile of gold and silver on a white background, serves notice that friendliness notwithstanding the bank has not been swayed from its appointed task of keeping money safe.

The form the bank eventually took was to a considerable extent suggested by certain physical requirements given the designer by the bank's president, George C. Parker. The need for 8500 sq ft of floor area, in addition to as much parking space as possible on the site, virtually dictated a two-level structure. The search for maximum accessibility of the drive-up banking window helped to set the circular pattern of this portion of the building. "To gain an open feeling," Architect Harkness explains, "steel was chosen for supporting columns because of its strength, ease of erection, durability and sense of slenderness."

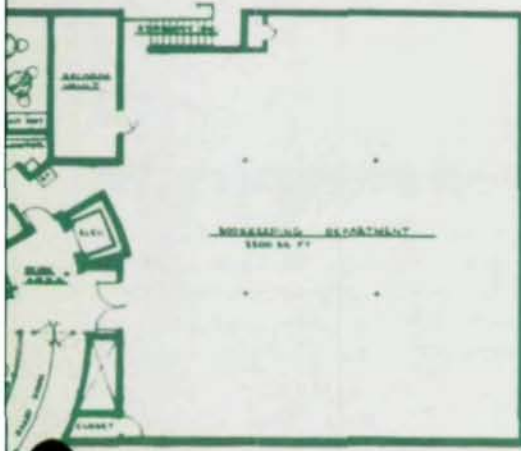
While the outer columns appear to support the roof and ramp, they are actually in tension; additional concrete footings were needed to counter-bal-

ance the interior loads. The roof beams are cantilevered from the window-wall ring of columns, except for some restraint from an eight-foot-diameter compression ring at the center which supports a plastic dome. These steel roof beams also help carry the second-floor beams through one-inch steel tie rods at the 18-ft diameter circular cutout, thus eliminating column congestion in the lobby below.

Architecturally, the rotunda effect provides excellent integration of the two floor areas and allows additional light and vertical spaciousness to the first floor lobby.

No detractor from the slender beauty of the exposed steel columns was allowed in the glass areas; glass stops were applied to the surface of the inner flanges.

Pittsburgh-Des Moines Steel Company was fabricator; the Harkness firm was coordinator of the many subcontractors as well as performing the work of designer. Building cost was \$19 per sq ft.



Pleated ceiling of the circular section is gold-colored acoustic plaster. Tie rods support the railing.

and an unusually interesting interior view to its left.

Further touches of friendliness are contributed by the walnut paneling used along with glass and exposed steel in the loan department, a metal fireplace in the board room, and a roof garden on top of the vault.

Almost as light-hearted as a carousel, the Community National Bank is an especially outspoken example of a fast-growing trend away from the old stern and sober look of banks (see **Modern**



QUESTIONS & ANSWERS

on the AISC Spec

During the four Specification Announcement Conferences held in January by AISC, many interesting questions were raised. Some of them are answered here with the hope that they will be useful to other designers.

Q. In Section 1.3.4. "Crane Runway Horizontal Forces." What speed is used in establishing the 20 per cent force? If not based on speed, what assumption was made?

A. The amount of lateral force for which crane runways are designed has no relationship to speed of operation of the crane. The largest lateral forces to which a crane runway might be subjected are produced when the crane is used to drag something across the floor so that the falls are inclined, and the force in the hoist falls contains a horizontal as well as vertical component.

Q. In cantilever construction where splices are provided at points of inflection, does the ten per cent reduction of moment apply?

A. This ten per cent reduction of moment was not intended to be applied to "cantilever construction," since redistribution of moment is not possible. Some moment will need to be taken at splices, since some migration of the point of inflection takes place as loading is increased to the ultimate. Therefore, some moment capacity is needed at the splice. At ultimate load the true or actual pin provides a different struc-

ture than where a continuous member is actually provided. No redistribution of moment takes place at elastic stress ranges. At ultimate load, however, the ten per cent reduction and redistribution of moment does take place, and plastic design should be the most logical approach. Any attempts to apply the ten per cent redistribution of moment are bound to show up inconsistencies, since elastic design is simply not as logical an approach as plastic design.

Q. In designing beams for 0.9 of computed negative moment, what modification should be made in the moment in the column in calculating F_b in formula (6)?

A. Same moment can be used in design of column as is permitted in the beam, provided axial stress in the column is less than $0.15F_y$. (This puts it in the category of a frame which could be designed plastically.)

If lateral loads are involved, the redistribution factor of 0.9 is not permitted, however.

Q. Are increases in allowable stresses due to continuity (Sect. 1.5.1.4.) and increases due to wind (or earthquake) cumulative?

A. No, the redistribution of moment allowed by using the 0.9 factor defined in Section 1.5.1.4.1 is applicable to gravity loading only.

Q. What is modular n for concrete for deflection in composite construction? Should this not be increased to allow for creep?

A. As for factor of safety, creep has very little effect. Creep affects the moment arm only, and it cannot change bending resistance greatly. When deflection is critical, however, creep is important. If temporary shores are used, dead load is supported by the composite section, and long-term stresses are involved; therefore, creep should be considered. If no shores are used, however, creep is no problem.

Q. How does the tension field concept apply to a plate girder with horizontal stiffeners? What will be the effect upon the new Specification?

A. Building codes do not provide for horizontal stiffeners in plate girders ordinarily. Horizontal stiffeners do not function well in making a girder web act like a Pratt truss. It would be like moving the chord of the truss in. Horizontal stiffeners contribute very little to tension field action.

Q. Can plate girder stiffeners be used on one side only to get tension field action?

A. Yes, but they require a larger area because of the eccentricity in thin-web tension field loading. This is provided for by the term D in Formula (10).

STEEL SCHOOL FOR THE HANDICAPPED

If architect B. Kenneth Johnstone had wished to do a routine job of designing Pittsburgh's Pioneer School for Handicapped Children, he could have provided the obvious list of expensive requirements for safety and usability — special drinking fountains, ramps, handrails, covered playgrounds and the like — and let it go at that. Instead, he strove to consider fully the problems of the mentally able but physically limited students and to give their school as many built-in advantages as possible.

- Avoid standard right-angle turns in halls; where halls change direction, substitute a gentler curve.

- Use broad window areas and huge airy spans.

- The framing must have no protruding legs and flanges to be dust-catchers or physical hazards.

- Design broad flat areas where color may be boldly applied.

- Achieve all this at lowest possible cost.

In turn, the design presented a challenge to consulting engineer R. A. Zern. He determined that the best economics within the architect's design could be obtained with a combination of steel and load-bearing brick walls — in fact steel was the only material suited to this design and type of structure. Choice of steel automatically enabled windows to be broad and spans spacious.

Next to join the search for the best means to accomplish the ends was the Guibert Steel Company, Pittsburgh fabricator. Among the solutions reached by Johnstone, Zern and Guibert Steel:

Instead of wide-flange sections, the rigid bent framing was made of standard shipbuilding channels automatically welded toe to toe. This not only precluded protuberances but also provided the desired flat surfaces for effective use of color.



The exposed steel framing for the Pioneer School utilizes specially welded channels that not only meet load specs but require little or no finishing material as well as permitting broader, better supported fenestration.



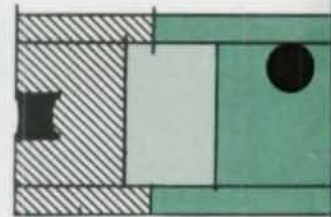
Welded corner supports on a 45-degree angle were designed to establish a gentle curve where halls change directions.

Special jigs were built and several innovations in welding techniques were made in order to deliver accurate, properly prepared and finished sections that could be exposed throughout and painted to provide a durable, attractive finish. The problem of warping, inherent in this sort of fabrication, was controlled by specially engineered cycling methods.

Glass curtain walls abutted directly on the steel sections, and the steel was exposed almost 90 per cent both inside and outside. The usual expense of such collateral materials as vertical studs, lath, plaster or other covering materials was thereby eliminated. The continuous and fast erection of steel saved labor and did not interfere with other craft installations.

Board of Education members feel that the cost of this school — \$851,295 for 40,250 sq ft, or \$21.15 per sq ft — is comfortably lower than a school with such advantages and innovations would otherwise have cost.

STRESS AND STRENGTH MATCHED IN BRIDGE GIRDERS



A pair of 305-ft end-span girders rolls into position. The long girders were assembled at the bridge approaches, then moved on a nest of steel rollers.

Three different steel grades, embracing the full strength range of commercially available structural steels, combine in an imaginative plate girder design for the Whiskey Creek Bridge near Redding, Calif.

The three-span, 870-ft bridge will carry traffic over a deep canyon near the old gold rush community of Whiskeytown in Northern California.

The 305-ft end-span girders cantilever 45 ft from the main piers to pick up a 260-ft suspended span. Total length of the main span is 350 ft.

The bridge comprises continuous plate girders of constant 12 ft-3½ in.

depth throughout most of their length. For negative bending zones over the two interior supports and for maximum positive bending zones in all three spans, the designers specified T-1 heat-treated constructional alloy steel (100,000-psi yield strength). For interior zones encompassing the inflection points and bounded by this steel, they specified ASTM A242 steel (42,000-psi yield strength in the thickness range used). And for low positive bending zones extending some 32 ft out from the abutments, they specified ASTM A373 steel (32,000-psi yield strength).

Maintaining a nearly constant girder

depth and uniform web thickness simplified fabrication and cut shop costs. If the bridge had been designed in one grade of steel, economy would have dictated haunching or heavy coverplating at the interior supports. Moreover, by keeping web depth and thickness constant, and flange thickness nearly so, the designers simplified welding and eliminated abrupt stress-raising changes in sections.

The design also cut the bridge weight substantially, thus facilitating shipping and erection. And the simple uncluttered lines make painting and maintenance easier.

Varying the steel strength to suit the stress enabled the designers to maintain a constant ¾-in. web, 12-ft deep. Flanges are 1¾-in. thick, except where flange thickness was increased to two inches.

If A373 steel had been used in this region instead of the heat-treated constructional alloy steel, flanges 5¾-in. thick would have been required. This would have drastically complicated both shop and field welding.

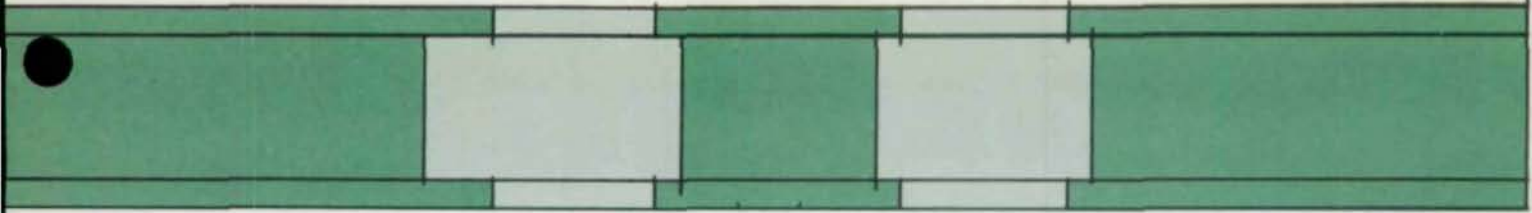





Diagram of one of the girders of the Whiskey Creek Bridge indicates how the various grades of high-strength steel were welded together. The smooth, unbroken lines improve aesthetics as well as reducing fabrication and future maintenance costs.

-  Heat-treated constructional alloy steel (100,000 psi)
-  ASTM A 242 (42,000 psi)
-  ASTM A 373 (32,000 psi)

Fabricated at the Orange, Texas, plant of the American Bridge Division of U. S. Steel, the girders were shipped to the site in lengths up to 109 ft. Each girder section — the four 305-ft end lengths (including the 45-ft cantilevers) and the two 260-ft suspended spans comprised six fabricated units. Steel workers assembled the two end girders on the high ground adjoining the abutments; they assembled the suspended spans on the canyon floor.

The crews first blocked up the girders on heavy timbers fitting them for camber before welders spliced them. In conformance with recommended practice for welding high-strength steels of these thicknesses, the welders preheated the spliced material and welded the joints with low-hydrogen electrodes. To stagger the flange and web welds, the designers detailed the splices with one set of girder flanges overlapping the web of the adjacent section, whose flanges are cut back.

Cross bracing and diaphragms were connected with high-strength bolts.



The 200-ft suspended span is hoisted into place by lead-line pull engines and a series of wire rope lifting falls.

A hoisting engine set on the bridge approach, a heavy steel block on the abutment, and a set of falls attached to the pair of end girders moved each 300-ton unit slowly out over the canyon. The girders rolled on three nests of steel rollers — one at the abutment, one at the main pier, and one at an intermediate falsework tower. When the girders were in position, crews jacked them up to remove the rollers and then lowered them onto the bearing shoes.

Giant concrete blocks placed on the top flanges at the abutment end of the girders helped to balance the load during the rolling operation.

In contrast to the end spans, which together took 9½ days to place, it took only eight hours to erect the center suspended span. Two 14,000-lb lead-line pull engines, each secured to the bridge approaches, and four lifting falls hoisted the heavy section from the canyon floor.

The hinged connections at the cantilever supports were made with nine-inch-diameter pins. Expansion joints, three inches wide, are located at these connections.

Bridge designer is the California Division of Highways. Lew Jones Construction Co. of San Jose, Calif., is prime contractor.



The Immeuble Consulaire, an eight-story office and apartment building housing 300 people in Agadir, is shown before and after the earthquake. A modern architectural facade proved to be no substitute for inadequate reinforcing at joints of walls and floors. The building was not braced to resist the lateral forces generated by the shock. Note the stack of successive floor slabs and precast beams.

At 11:41:14 PM on the night of Feb. 29, 1960, an earthquake devastated the Moroccan city of Agadir. Of this port city's ten areas, three were completely destroyed and five were 60-90 per cent demolished. The quake reached an intensity of 5¾ on the Richter scale — a mild one compared to other devastating earthquakes of history — but in only 15 seconds 12,000 persons, one-third of the population, were killed, and an equal number injured.

Within three weeks a team of American engineers were on the scene: Professor Ray Clough of the University of California, a widely known author in the field of earthquake engineering; R. W. Binder of Bethlehem Steel Company, Pacific Coast Division, who is a director of the Earthquake Engineering Research Institute; W. G. Kirkland, chief of engineering division and assistant vice president of American Iron and Steel Institute and T. R. Higgins, director of engineering and research for the American Institute of Steel Construction.

They visited all parts of the city, carefully examining buildings in conditions ranging from utter ruin to relatively

TEACHES

minor damage. They assisted in one of the most complete studies of earthquakes ever assembled, and the results of their study are reported in **The Agadir, Morocco, Earthquake**, a 112-page book published by the American Iron and Steel Institute on March 1, 1962, the second anniversary of the quake.

The authors noted one glaring, inescapable fact: Among the buildings most damaged there was a predominance of shoddy, weak construction, such as masonry with poor bond or no bond at all. In some they observed a complete lack of ties between walls and floors or between other structural elements.

Although reinforced concrete construction was in ample evidence on the newer structures, joints between columns and beams were unable to withstand moment and shear stresses created by lateral forces.

By contrast, in the few instances where the designer had provided reinforcement and continuity and some resistance to lateral forces, the damage was much less severe, even though some were in a high intensity zone of the earthquake.

In reporting the results of their study, the engineers make it abundantly clear that the tragic loss of human life and the tremendous structural damage could have been prevented or materially reduced through structural materials, techniques and engineering knowledge available. By extension, all structures should be engineered for continuity and bracing for lateral forces.

Reasonable economical provisions against earthquakes must take advantage of energy absorptive ability in a building. Thus a structure might be designed and built to resist accelerations elastically without damage and large accelerations non-elastically — with damage, but without great peril to occupants.

As an example, there were several steel-framed structures at the Agadir waterfront — sheds, warehouses, and

boring structures.

"Our structures must be ductile rather than brittle, flexible enough to permit movement without internal violence. To do this, the designer must give proper attention to the types of forces generated by earthquakes, the influence of relative rigidities of the structural elements of a building in distributing these forces throughout the structure, and to the utilization of materials which have adequate elastic strength and which can continue to function after sustaining significant inelastic deformations."

The authors deal with the calculation of the dynamic responses of flexible structures. First, the simple, elastic, one-mass system such as a one-story building; then the multi-story structure. In a sample analysis of a multi-story building, they proceed from weight and elastic properties to computer-determination of the building's vibration mode shapes and periods, its base shear response, the effective forces acting at each story in each mode, the maximum modal shears and the total shear response.

The maximum base shear is found to be 19 per cent of the structure's total weight. This, they explain, does not mean that typical lateral-force codes giving base-shear coefficients of three to five per cent are departing from the conservative. It simply means that buildings often absorb lateral forces through non-elastic yielding and cracking and thus prevent energy from building up to the magnitude calculated in the example.

The authors conclude that "there is every reason to believe that structures designed in accordance with the lateral-force provisions of a modern building code (such as that recommended by the Structural Engineers Association of California) would perform very satisfactorily in earthquakes of the intensity experienced in Agadir."

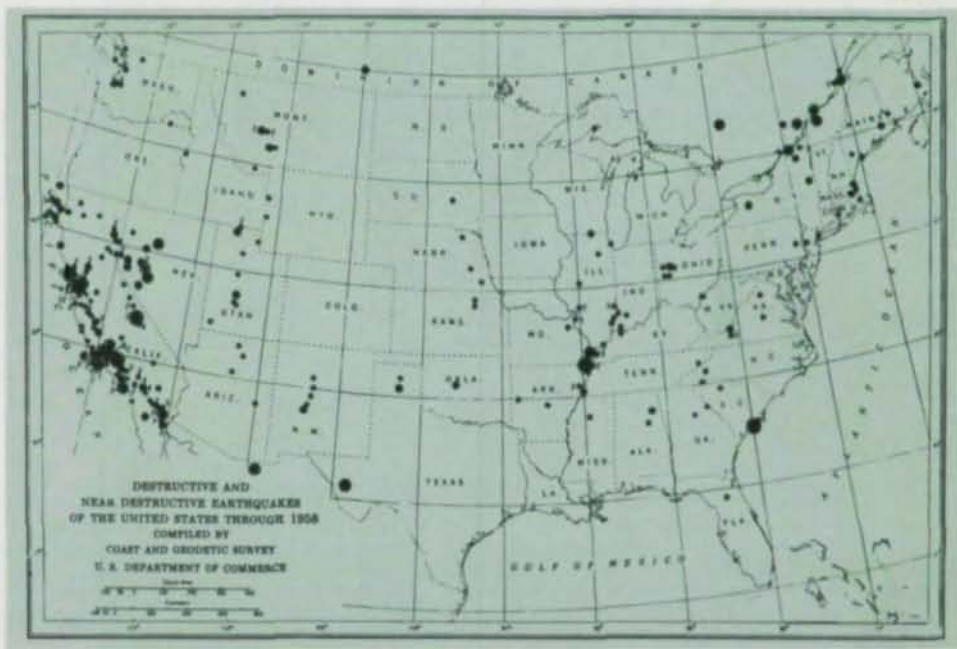
Published in a limited edition, The Agadir, Morocco Earthquake has been distributed to structural engineers, building officials, schools of engineering and university libraries where it is readily available to every serious student of engineering intent on designing structures that will be safe under earthquake conditions. Copies are also in AISI and AISC offices throughout the United States.

STRUCTURAL LESSONS

cranes. Except for the damage which occurred when a crane toppled over on a warehouse, these lightweight, well-braced structures were practically unaffected by the earthquake. Sheds settled with the subsidence of uncompacted fill, but remained structurally sound and showed no vibrational damage.

"We can neither predict nor prevent earthquakes," Mr. Kirkland says. "We can, however, design buildings that will absorb and redistribute shock among members; buildings that will 'roll with the punch;' buildings that will protect occupants against the collapse of neigh-

Earthquakes also happen in the United States. Destructive or near destructive earthquakes have had their epicenters in 40 of our 50 states. In fact, no place on earth is free of such danger. One of North America's strongest earthquakes occurred in Charleston, S. C., in 1886 — in an area believed to be virtually immune.



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