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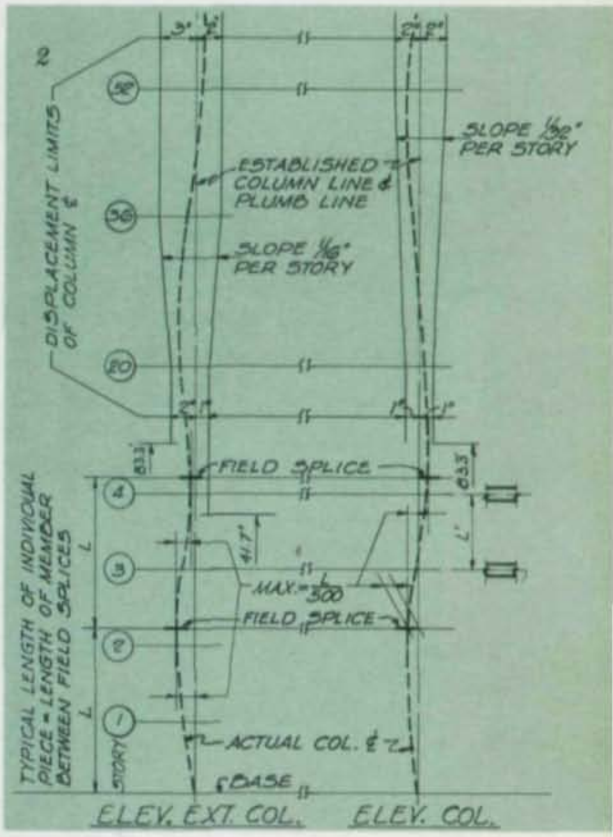
MODERN STEEL CONSTRUCTION



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Steel

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AN INVITATION

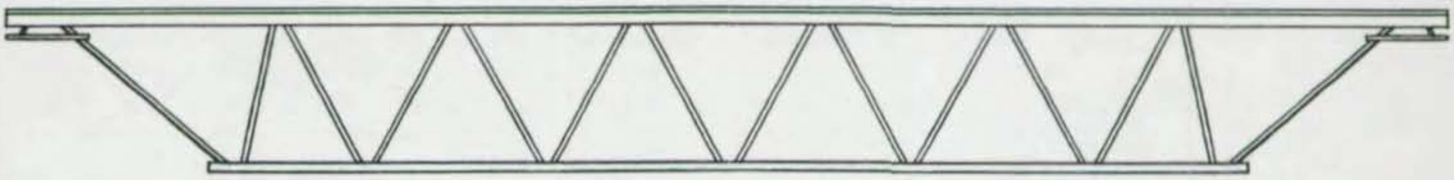
Economy and versatility combined with safety and strength are the major advantages of structural steel, and the quickening pace and results of research are producing better ways to design in steel. Whether the results come from the laboratory, the experimental and probing mind of a designer, or the actual construction of a bridge or building, the American Institute of Steel Construction is attempting to dessiminate useful design material as quickly as possible.

For future issues of MODERN STEEL CONSTRUCTION, *we would like to invite architects and engineers to send in manuscripts, photographs and sketches describing their structures, methods or just ideas that they feel would be of interest to our 20,000 readers. It is our hope that we can provide a forum for new thinking and useful information. The design professions, clients and the public will benefit through more economical and better structures.*

A REMINDER

Architectural Awards of Excellence Entries for buildings completed or occupied during 1962 are due in the New York office of the AISC by May 15, 1963. Announcement of the winning architects will be made about July 1. A brochure containing entry rules was mailed the end of March, and additional copies are available through our regional offices or from New York.

HOW QUIET ARE



STEEL-FRAMED FLOORS

John G. Hotchkiss,
Senior Regional Engineer

Impact noise-control is a major problem in a multi-family dwellings due to the increased noisiness of our present-day environment. Impact noises are the most annoying kinds of noise. Such unwanted sounds as thumping footsteps, a fallen object, a neighbor's TV set, or a noisy dishwasher overhead can readily be transmitted by vibration through the floor and ceiling if adequate measures are not taken to reduce the impact sound waves.

Perhaps because little information about impact noise is currently available in this country, certain misconceptions and fallacies have arisen. One of these involves the use of steel bar joists in a floor assembly. Such construction is often erroneously alleged to be noisy. This can be readily disproved. Test results on floor assemblies both here and abroad show that it is only the **surface** of the floor that is critical for noises due to **impact** and not the structural elements. Floor treatment is independent of the structural assembly.

Since sounds in a room may be transmitted by two different means, they accordingly have been classified as impact noise and airborne noise. (Fig. 1) The more familiar airborne noise is produced by the TV or hi-fi set, the human voice, or a musical instrument radiating sound waves through the air in the room above. When these waves strike the floor, it vibrates and in turn radiates sound into the air in the apartment below.

The problem of insulating **airborne** noise is minimized by using structural assemblies that are massive or are composed of heterogeneous materials. Changes in the floor surface, as mentioned above for impact sound isolation, have little effect. By contrast, impact sounds are generated by objects striking or scraping across the floor. Noises such as a dishwasher, an object dropped on the floor, a bathroom shower, furniture being moved, a rotating machine, or footsteps, are sounds that are passed on to the floor by direct mechanical contact. Of importance is the fact that the sound thus radiated within the floor continues on radiating waves from **both** the top and bottom surfaces of the floor. Thus certain floor assemblies designed to control impact noise will also help control the transmission of airborne noise. The converse case is very unlikely to occur.

The United States is one of the few countries in the world which up to recently has had no noise-control criterion. Recognition of such a standard has been established by England, Sweden, Germany, Austria, Bulgaria, Canada, Denmark, Finland, France, Netherlands, Norway, Czechoslovakia, and the USSR. Due to the growing number of complaints of inadequate sound isolation in multi-family dwellings, the Federal Housing Administration in January of this year prepared the first American impact noise-level criterion, along with a compilation of noise performance data. This publication, FHA NO. 750, *Impact Noise Control in Multi-family Dwellings* was the result of interviews with thousands of tenants as well as a review of many foreign codes.

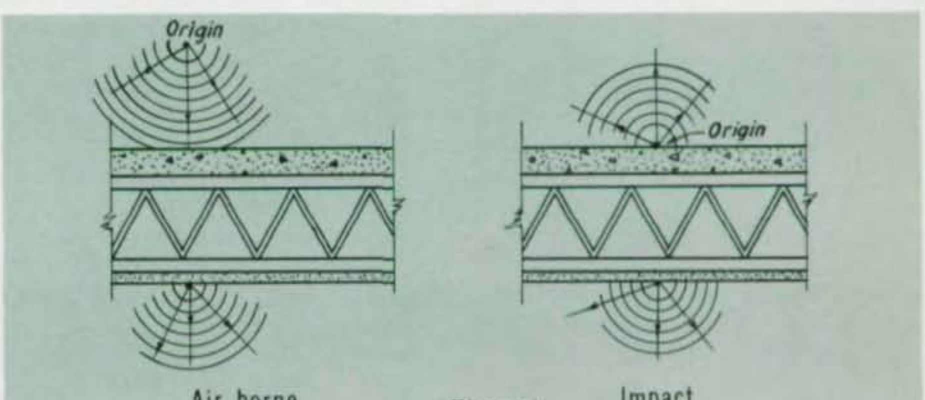


Figure 1

Any criterion for noise-control must acknowledge the possible existence of ambient noises: the quiet background of urban environment, the background noise of flowing traffic in the cities or simply an air-conditioning machine. These ambient noises may help mask the sporadic, unwanted sounds coming from a neighbor's apartment. One must also consider how much isolation can be provided without greatly increasing the cost of construction. Since years of construction without code restrictions for noise-control produced inadequate isolation, new practices will cause some increase in construction cost. Significantly, however, the amount of this increase is small if noise-control is considered in the early design stage. Addition at a later date often is costly.

For multi-family dwellings the FHA has recommended values for the maximum impact sound-pressure level in floor assemblies. It is a criterion of acceptable, but not excessive, isolation against impact noise. By comparison with this criterion, impact performance characteristics have been classified by type of floor construction and the ability to provide, or not provide, the recommended isolation. A single number impact noise-rating (INR) is used. A floor assembly rated INR = 0 or above is acceptable. The larger the INR the better the floor. It is sometimes advantageous to know the ranking of a floor construction; viz., does it barely exceed the minimum or does it greatly exceed the requisite? A negative INR rating, for example INR = −4, means that an improvement of 4db would be required to reach acceptability (INR = 0). A rating of + 26 – the bar joist assembly shown in the Table – means that the impact isolation rating has so much tolerance that a reduction of 26db would be required to bring it back to zero. Such a design would be far more than required for even a luxury apartment.

According to FHA, an apartment in a quiet area should have a floor construction better than normal, INR = +5. A poorer construction (INR = −5) may be tolerable in noisy urban districts where ambient noises produce a masking effect. This criterion, as suggested, takes into account an ambient noise for typical apartment construction as given in the guide published by the American Society for Heating, Refrigeration and Air Conditioning Engineers.

The Table, "Impact Noise Control Tests," shows various assemblies with the structural elements, floor finishes, ceiling finishes, and in the boxes, the INR rating. The testing authorities are also given. Obviously this listing is far from complete. However, the few selected serve to demonstrate the noise-control attainable by careful choice of floor finish. It should also be observed that the softer, resilient materials are more effective in the isolation of impact noise. Take the case of the steel bar joist floor. The highest rating (INR = +26 tested at the Bureau of Standards), published by FHA, requires a ⅜-in. nylon carpet on a ¼-in. foam rubber pad. The structural assembly is typical construction used in multi-dwellings where a fire-resistance rating of three hours is required. For this fire rating, gypsum perlite plaster would be used in place of the sanded gypsum plaster tested. Since the composition of the ceiling finish has little or no affect on the INR rating, such a substitution is valid. The second highest INR rating published for bar joists (INR = −2) shows ¼-in. cork tile cemented to the concrete with linoleum paste. A change from INR = +26 to INR = − 2 is very significant, yet no change was made in the structural assembly.

Now consider the INR ratings for the flat concrete slab construction, which is a typical framing system used for multi-family dwellings. The highest rating (INR = +12) published shows a cocomat floor cover or three-eighth-inch carpet laid over a three-quarter-inch plaster screed. If one-eighth-inch linoleum is cemented to a two-inch concrete screed which in turn rests on a one-inch glass fiber blanket atop a five-inch concrete slab, the INR = +1. Again this is a significant change amounting to a reduction of 11db. The bare concrete slab (INR = −17) was included in this Table to indicate how transparent such floor construction is to the transmission of impact noises. The one test shown for concrete joists was included to show that an elaborate construction – a floating wood raft on a glass fiber blanket – only provided an INR = +2. Such construction would be ruled out as too costly. Furthermore, this assembly does not meet the requirements for a three-hour fire-resistance rating.

Thus far only impact noise-control has been discussed. Insofar as floor construction is concerned, impact isolation is of far more importance. However, there is test data currently available covering sound transmission of air-borne noise through floors. Disregarding any floor treatment required for impact noise-control, the typical steel bar joist floor with a plastered ceiling has sound reduction ratings in the order of 55-65 db.* Good American practice according to Bolt & Newman (well-known acoustic consultants), suggests 40-50 db. A bare concrete slab has a 41-43db rating.

Some further comments regarding noisy dwellings should be mentioned, because noise-control involves many more problems than that of the floor alone. A common problem concerns the control of transmitted sound over and around partial-height partitions. In such cases, particular attention must be directed to the elimination of serious air leaks. Air-borne sounds originating in one apartment may pass through a suspended ceiling into the adjacent room where the partition extends only to the suspended ceiling. Other typical leaks between apartments very often come from plumbing fixtures, radiators, lighting conduits and back-to-back outlets.

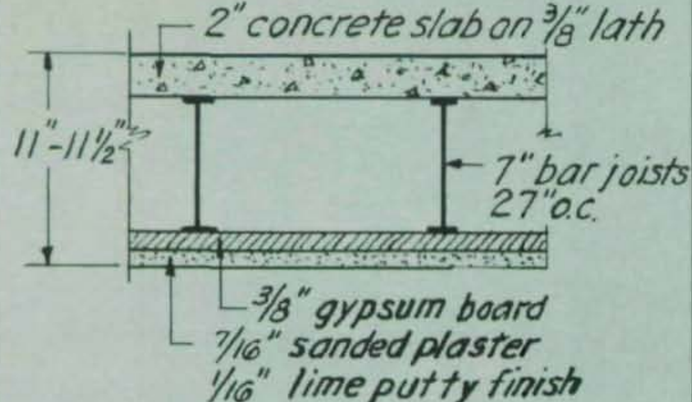
In such a short discussion it is impossible to cover the complex problems confronting the architect or acoustical engineer who attempts to control noise within cost limitations. Nevertheless, it is recommended that such allegations as "noisy" bar-joist floors should be re-evaluated in light of the new test results.

Such tests have been conducted by the National Bureau of Standards and the Electrical Research Products, Inc. (Western Electric Company).

IMPACT NOISE CONTROL TESTS

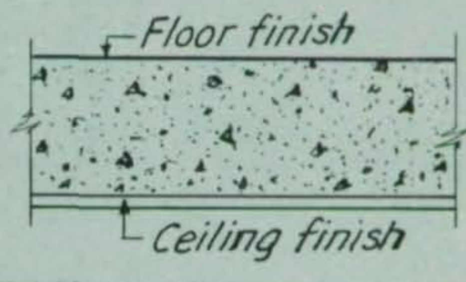
Source: Impact Noise Control FHA #750

STEEL BAR JOISTS



3/8" Nylon carpet (¼" pile) on ¼" rubber foam pad. Authority: Nat'l. Bureau of Standards. Remarks: With ½" gypsum perlite plaster and 1" 20 ga. mesh this assembly has a 3 hr. fire rating.	+26
¼" Cork tile cemented to concrete with linoleum paste. Authority: Na'tl. Bureau of Standards.	-2
1/8" Vinyl asbestos tile cemented to concrete with linoleum paste. Authority: Nat'l. Bureau of Standards.	-10

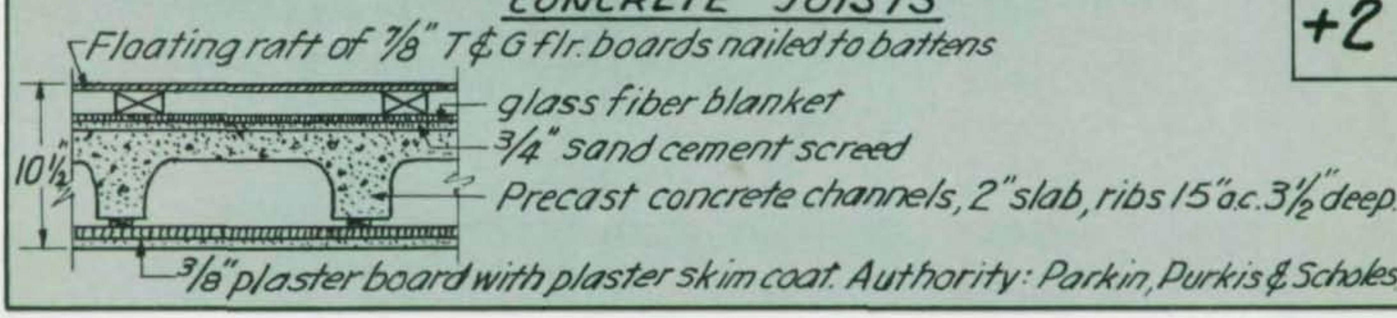
FLAT CONCRETE SLAB



Cocomat floor cover or 3/8" carpet on ¾" plaster screed on 4½" concrete slab. 3/8" plaster ceiling. Authority: Research Inst. for Public Health Engr'g. Netherlands 1950.	+12
1/8" Linoleum cemented to 2" concrete screed on 1" glass fiber blanket on 5" concrete slab. ½" plaster ceiling. Authority: Parkin, Purkis & Scholes. London, 1960.	+1
Bare concrete slab or 5/8" pitch mastic or 5/8" composition on 6½"–9½" concrete slab. ½" plaster ceiling or bare. Authority: Parkin, Purkis & Scholes. London, 1960.	-17

CONCRETE JOISTS

+2



APARTMENTS GO STEEL IN WASHINGTON

One of the most significant trends in the construction industry is taking place in the nation's capital. Architects and engineers in the area are providing their clients with probably the most economical yet handsome apartment buildings anywhere in the country through welded, continuously-designed steel frames.

The chart on the opposite page tabulates some statistics for nine projects which are typical of the more than two dozen, seven to twelve-story apartment houses which have been built in and around the District of Columbia in the past several years. As a result of the economy inherent in welded continuity, other framing schemes, widely used for many years, are becoming obsolete for multiple dwellings in the area. Construction is less expensive, and the steel frames are being erected as fast as other types of framing.

By designing steel as a continuous frame (as most engineers design concrete), savings up to 20 per cent in weight are possible. In addition, the detail material is reduced to about one per cent. The combined weight of columns and beams averages about 5.5 pounds per square foot with an average cost of 15¢ per pound. The actual unit-weight depends on the particular span and bay area. The absolute minimum in weight and floor depth can be achieved by using cover plates in the negative moment areas and designing the beam for the usually smaller positive moment. This approach was used by the structural engineer for the Congressional Apartments (Project 9).

Washington designers are also taking full advantage of the new AISC Specification to achieve light steel weight. A36 steel is used for basic steel framing, but A441 and other high-strength steels are also specified for lower columns. A six-story building is presently being designed entirely with 50 ksi-yield steel. For Crystal House, Arlington, Va. (Project 8), the engineer designed with 50,000 psi steel in the lower columns to save weight and maintain minimum column design. K-trusses were used as the most economical method of carrying wind loads to the foundations.

Figure 2

Whitehall Apartments, Bathesda, Md., have spans of 28 to 30 ft. Balconies are framed with exposed H-sections.

Figure 3 ▶

Most of these welded structures have been erected with the aid of Saxe clips. These specially forged connection units are shop-welded to the beams and columns and hold the connections rigidly in place while the field-weld is made. Temporary seat angles and erection bolts are therefore eliminated. Welding is done by certified welders and is inspected by independent inspection service agencies. Cost for this welding inspection service runs about two to three dollars per ton.

Architects in the capital area are not sacrificing beauty for economy. The Whitehall Apartments in Bethesda (Figure 2) recently won for its builders the Oliver Owen Kuhn Cup for "an outstanding example of luxury-type, high-rise apartment building, constructed of the finest materials and of a design forward-looking in its approach yet contemporary in its appeal."

The higher unit weight was due to longer spans (28 to 30 ft.) and a considerable number of balconies.

The Horizon Apartments (Figure 5) is another handsome but very economical structure. Floor beams are ten-inch wide flange sections fully welded for continuity which serve as wind beams and main-load carrying members.

The Park Maple Apartment House (Figure 6) was designed with an unusual balcony structure. Rectangular steel tubes run the full height of the building and support the balconies at each floor. This luxury building, equipped with air-conditioning, swimming pool and parking facilities, was occupied nine months after ground was broken.

FIGURE 1

Project	No. Stories	No. of Units	Floor Area Square feet	Steel Wt. psf	Joist Wt. psf
1. Park Maple Apts. Tokoma Park, Md.	12	135	157,000	6.55	1.91
2. Park East Apts. Washington, D. C.	8	88	60,000	5.75	2.67
3. Whitehall Apts. Bethesda, Md.	11	209	265,100	7.32	3.08
4. Horizon Apts. Arlington, Va.	9	77	105,400	5.2	2.5
5. Abingdon House Washington, D. C.	7	82	88,700	4.65	2.1
6. Suburban Tower Silver Springs, Md.	12	174	202,800	6.3	2.8
7. Summit Hill Apts. Silver Springs, Md.	11	123	189,000	6.4	2.85
8. Crystal House Arlington, Va.	12	426	435,000	Total 9.8	
9. Congressional Apts. Rockville, Md.	8	300	358,000	5.56	3.60

Architects
Berla Abel, Washington, D. C.: Project 1
Werner-Dyer Associates, Washington, D. C.: Project 2
Cohen, Haft AIA & Associates, Silver Springs, Md.: Projects 3, 6, 9
Sheridan Behm, AIA, Arlington, Va.: Project 4, 5
Edmund W. Dreyfuss & Associates, Washington, D. C.: Project 7
Edwin Weihe, Washington, D. C.: Project 8

Structural Engineers
Michael K. Johns, Washington, D. C.: Projects 1, 2
H. L. Keller & Associates, Silver Springs, Md.: Projects 3, 6, 7, 9
Horatio Allison, Rockville, Md.: Projects 4, 5
Heinzman & Clifton, Washington, D. C.: Project 8

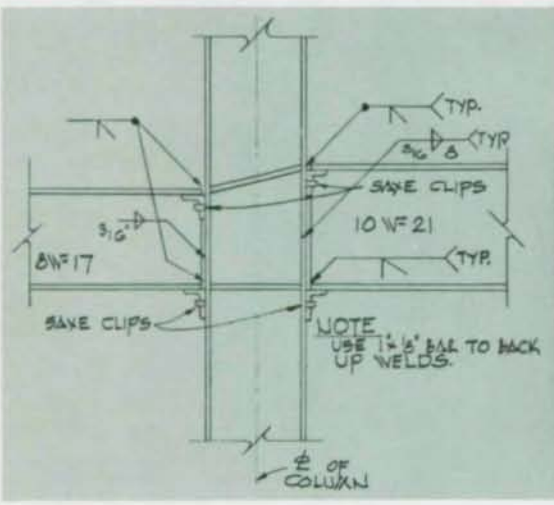


Figure 4

Figure 5

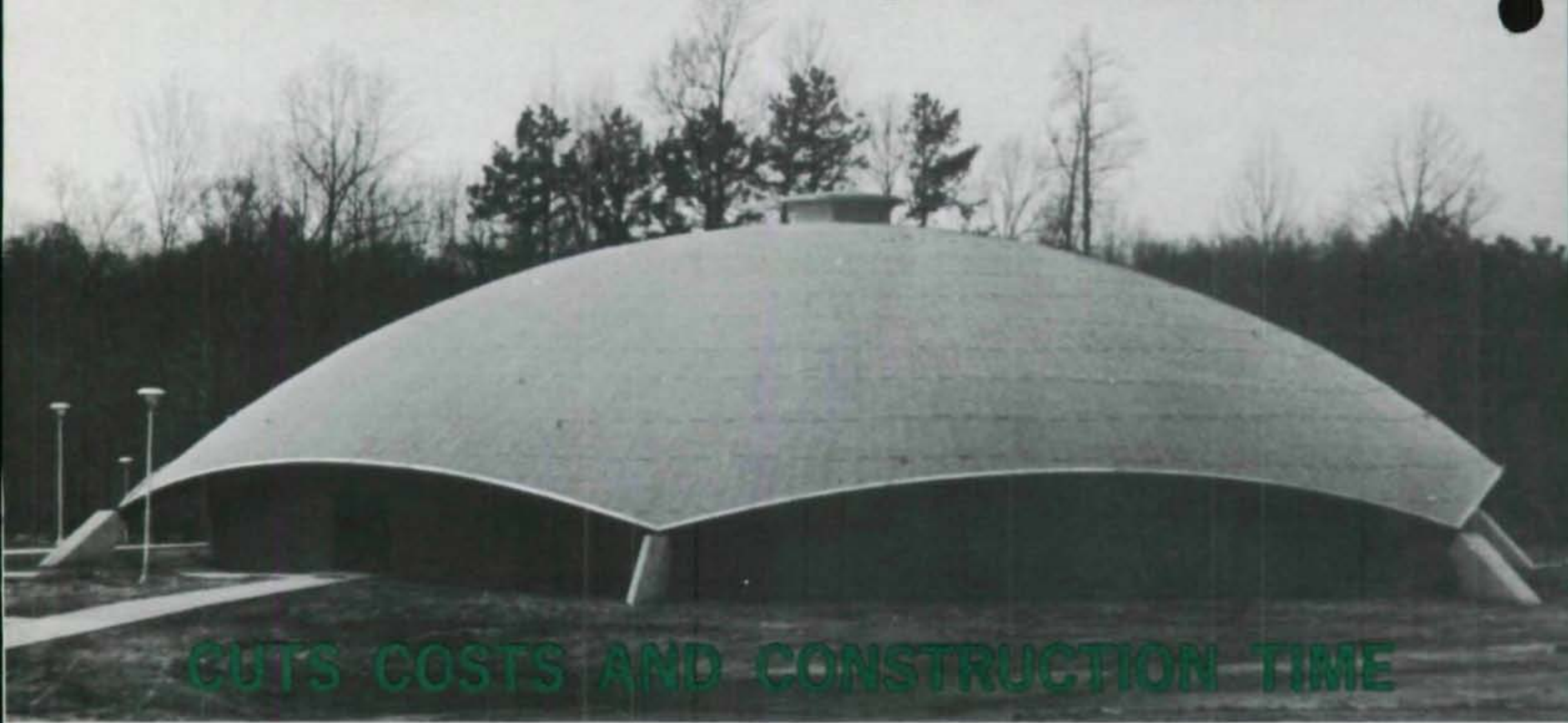
Horizon Apartments, Arlington, Va., also has cantilevered balconies. Figure 4 shows the simple, inexpensive detail used by the engineer for the balcony framing.

Builders of apartment houses (both public and speculative) have been experimenting for years to achieve low cost yet attractive and serviceable designs. The significance of the experience in Washington is that examination by engineers of continuity design, utilization of economies outlined in the new AISC Specification, and framing where applicable with high strength steels meet these requirements.

Figure 6

Steelwork for the Park Maple Apartments, Tokoma Park, Md., was erected in 7½ weeks. Building was occupied nine months after ground was broken.

STEEL GEODESIC DOME



CUTS COSTS AND CONSTRUCTION TIME

by E. E. Hanks
Technical Editor

The John T. Williams High School Gymnasium, recently completed in Charlotte, N. C., is a geodesic dome covering 13,000 sq ft and costing $154,000, or about $12 per sq ft.

The steel frame projects over a brick wall which encloses the gymnasium. Inside diameter of the closure wall is 128 ft, with an area of 13,000 sq ft. Outside diameter of the frame at minimum overhang is 132 ft, and 154 ft at maximum overhang at the six supporting concrete piers. Height above tension ring is 22 ft-2 in., and 32 ft-2 in. above the finished floor.

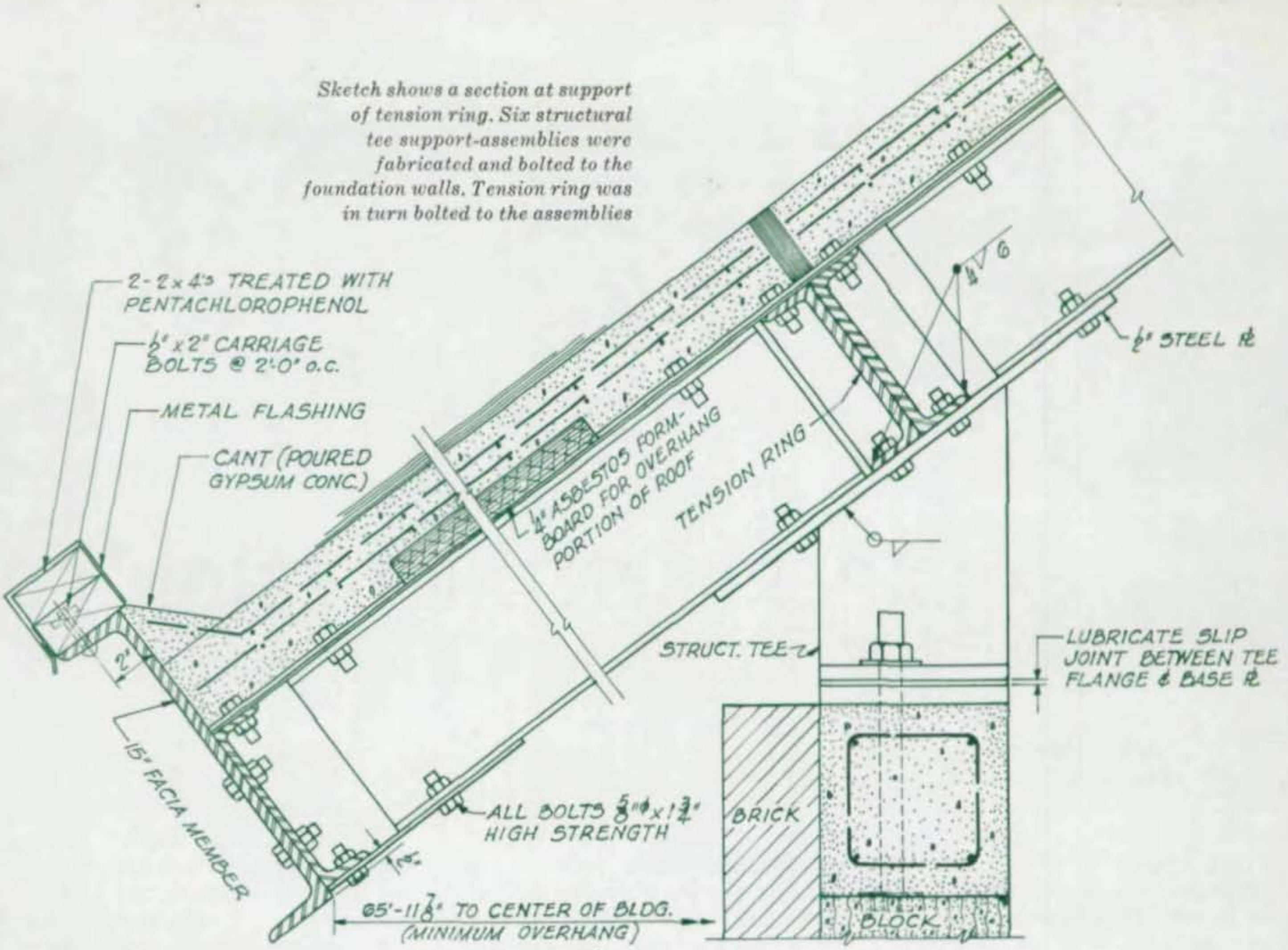
The design of the dome combines structural steel framing with poured gypsum deck on bulb tees, with the steel exposed on the underside for a pleasing architectural effect and to provide noise-damping coffers.

The designer estimates that the dome design and construction methods saved about 17 per cent over the conventional rectangular building totaling several thousand dollars in construction costs. Cost of erection was low, as a steel geodesic dome-frame supports itself as it is being erected, so no falsework is needed.

The frame is composed of 378 trihedrons made of 8 B 10 steel sections with arched fascia members of the overhang of 15 [33.9. The tension ring is made of two 8 [18.75, first welded together back-to-back and then butt-welded around the circumference of the closure wall. Members of the trihedrons were slightly curved to conform to the curvature of the dome. Control radius of the sphere is 105 ft. Vertical component of reaction is carried by the bearing (closure) wall.

Weight of the structural steel (grade A-7) was about 57 tons or eight pounds per square foot including overhang area. This was the first geodesic dome for the fabricators, Southern Engineering Company, Charlotte, N. C., and personnel had some misgivings and apprehension about tackling this job. However, they report that no appreciable difficulties were encountered in the preparation of the shop drawings or with the fabrication and erection of the steel. Field connections were made with high strength bolts. No falsework was needed.

The geodesic dome is an excellent example of how structural steel, due to its ductility and pliancy, can be shaped



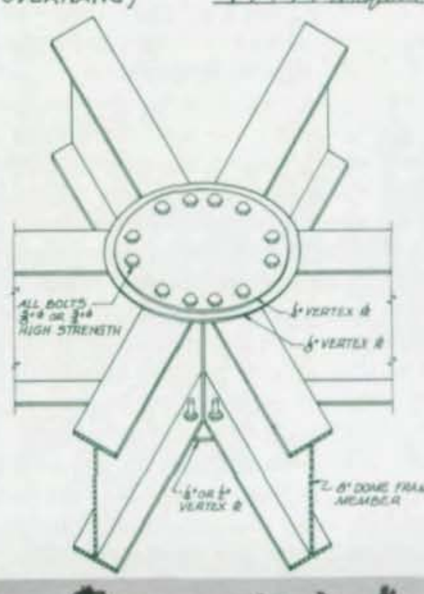
Sketch shows a section at support of tension ring. Six structural tee support-assemblies were fabricated and bolted to the foundation walls. Tension ring was in turn bolted to the assemblies

to fit any form or shape. Designers can take advantage of steel's inherent strength, economy and compactness regardless of the contour of the structure.

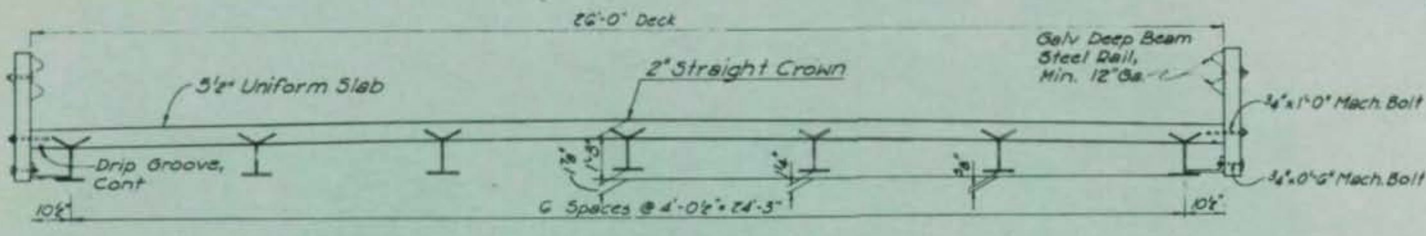
Architects were Biberstein, Bowles, Meacham & Reed, Charlotte, N. C. Robert H. Pinnix, Inc., Gastonia, was the general contractor.

The design was by Synergetics, Inc., consulting engineers, Dr. M. E. Uyanik, structural consultant, of Raleigh, North Carolina. The firm specializes in the design of structural steel geodesic domes. The geodesic dome has all its members as arcs of greater circles. The center of all the greater circles is the center of the sphere. It is a comparatively new concept in steel design, and the firm has designed several structures.

In addition to the Charlotte dome, they include the Union Tank Car Company Plant (384-ft diameter), Baton Rouge, La. – the first one built from plans developed by R. Buckminster Fuller; the Walt Whitman High School Field House (157-ft clear span), Bethesda, Md.; and the American Society of Metal Research Center dome, Cleveland, Ohio.



Sketch at left shows typical connection of dome frame members. Photo below shows the completed geodesic dome before roofing was installed.



TYPICAL ROADWAY SECTION

EXPERIMENTAL DESIGN FOR

by R. S. Delameter
Consulting Engineer

Our office has recently developed a type of composite-beam design which we believe will prove to be practical as well as economical for constructing short-span, continuous-beam bridges. The design uses a large structural tee with the flange serving as the lower flange of the composite beam, and a concrete slab anchored to and partly encasing the upturned edge of the tee web to form the entire top flange. The design, we think, may be of some interest to anyone having occasion to design short-span bridges.

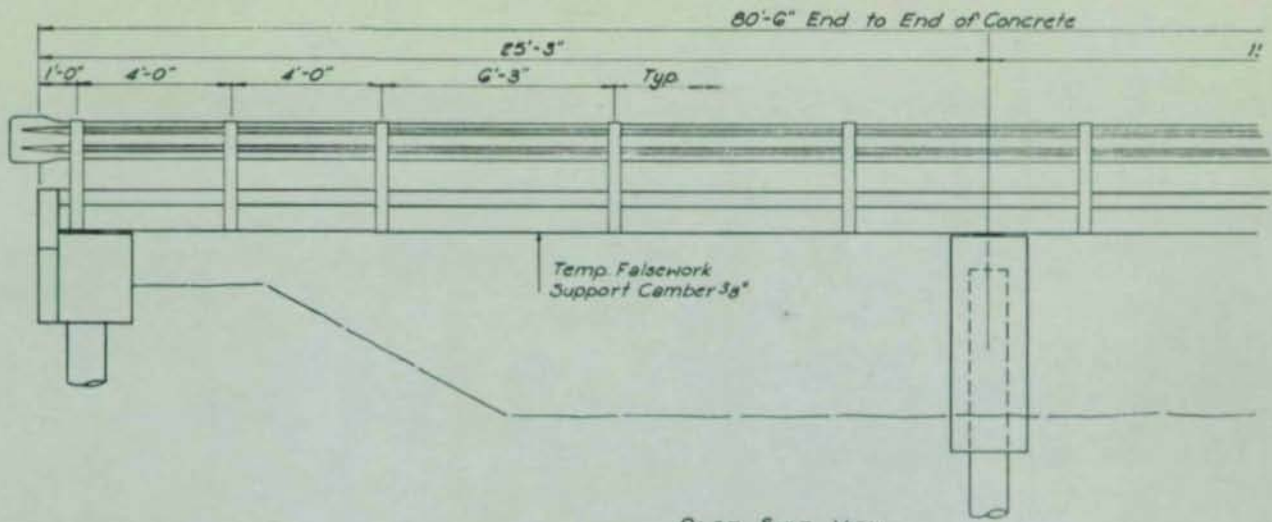
Since most county highway departments traditionally have minimum funds to maintain the kind of county road systems which the public has come to expect, maximum economy in construction becomes almost a primary requirement. Adequate structural design is, of course, the first requirement of any bridge-design system, and the cost of maintenance certainly must be taken into account. To achieve maximum economy we have tried to take advantage of every design method which offered a possibility of reducing construction and maintenance costs without sacrificing the load-carrying capacity of the bridge. Some years ago we started using composite design on our rolled-beam steel bridges and found that it considerably reduced weight of steel without materially increasing the cost of construction. This method of design has since been adopted as a standard by the Secondary Road Department of the Kansas Highway Commission.

Our objective on shorter spans was to design a bridge suitable for use on local roads where traffic generally is quite light, mostly local in nature, but where occasional heavy loads may be expected and must be carried without damage to the bridge. It was desirable to keep the total depth of deck to a minimum and to use a continuous-beam design, not only for economy but also to eliminate the maintenance problem which usually accompanies joints in the deck. We decided to try a fully composite beam with the concrete slab forming the top flange throughout the length of the bridge. We adopted for stringers a structural tee-section with the web turned up, and the concrete slab encasing the top inch of the web and anchored to it by connectors consisting of stud-welded rods bent up enough to be fully embedded in the slab. Longitudinal steel in the slab will provide tensile reinforcement required to carry the entire negative moment at the piers.

We set the stringer spacing to give what we believe is maximum economy and simplicity in forming. The deck can be formed by setting full-width, four-foot plywood panels which will just fit between the beam webs and which can be supported on transverse four-by-fours blocked up off the bottom flange, thus eliminating any cutting of form boards or the use of any complicated form-hangers. It should be possible to reuse the form panels several times, and both we and the county engineer feel that this is about the simplest forming system that could be devised for this type of bridge.

The width of the roadway is really an outgrowth of the beam spacing. We started with a roadway width of 24 ft, pretty much a minimum for county roads in this state; however, using seven beams to allow for good anchorage, we wind up with a 26-ft slab, and, in effect, figure that we have a foot and a half of extra roadway at very small cost, which is more or less a bonus and certainly won't be any disadvantage. If it were desirable to build a narrower roadway one or more beams could be eliminated, reducing the deck width by four-foot increments.

For the short spans involved (30-ft main spans) we decided to see if we could achieve the "ultimate" in com-



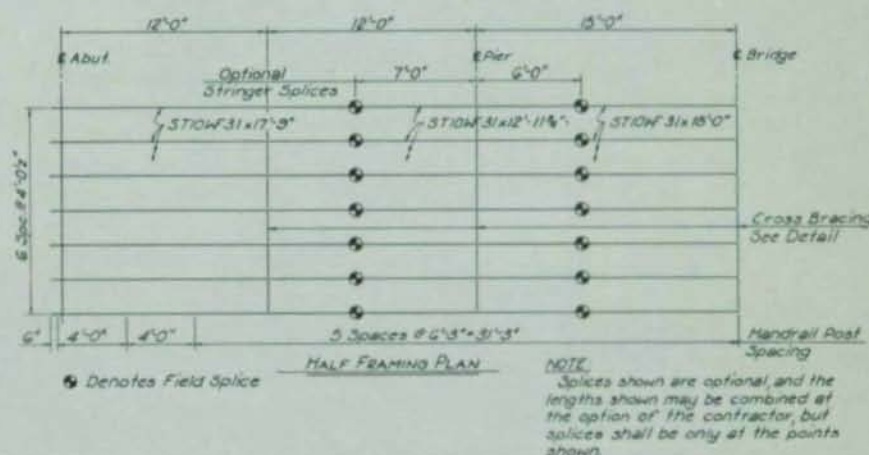
PART SIDE VIEW

SHORT-SPAN BRIDGES

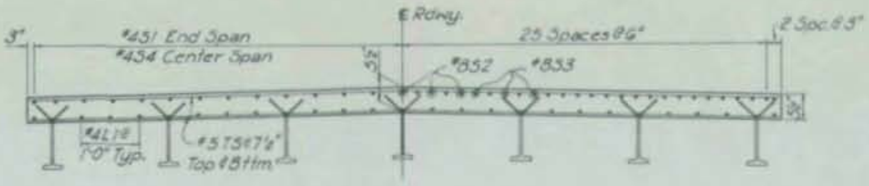
posite construction; to have no steel compression flange whatever. We found that this wasn't too much of a problem in the positive moment section. A large structural tee would carry the dead-load moment without much trouble; the only question was what kind of connectors to use to develop bond between the upper edge of the tee section and the concrete.

We considered several alternatives and finally concluded that the simplest would be stud-welded connectors, as we have used for several years on flanged sections. However, the ordinary headed-stud would barely be covered by the slab. We decided to use a stud-welded rod long enough so it could be bent up and fully embedded in the slab. The manual operation of bending up the stud after welding doesn't seem to be a major problem, and it does provide probably the simplest and fastest means of putting a connector on the stringer, and do so without excessive welding which might distort the comparatively light members we are using.

In order both to decrease costs and to help keep the deck clean we are using a deep-beam-type steel guardrail to serve as combination curb and handrail. The rail is supported on posts bolted directly to the edge of the slab and to

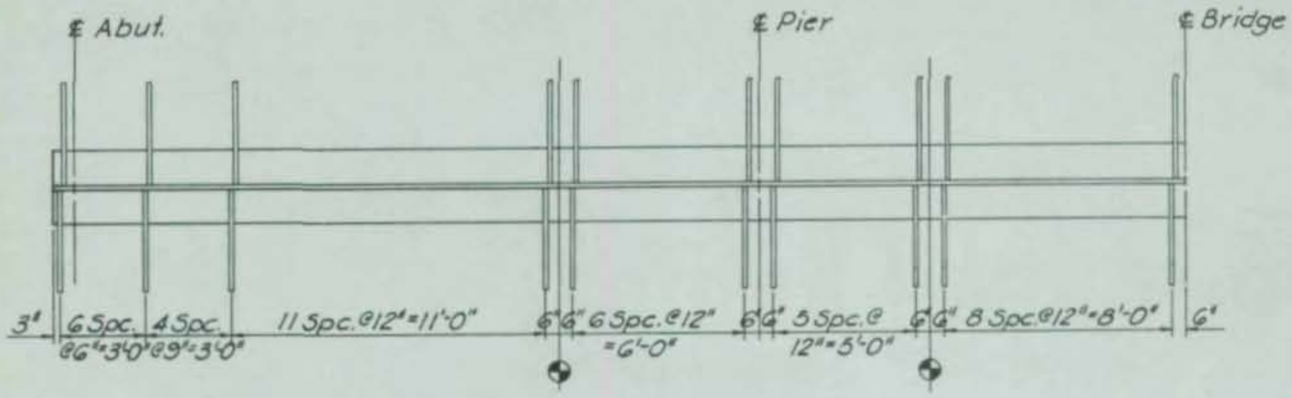


SLAB REINFORCING PLAN



HALF SECTION AT MID SPAN

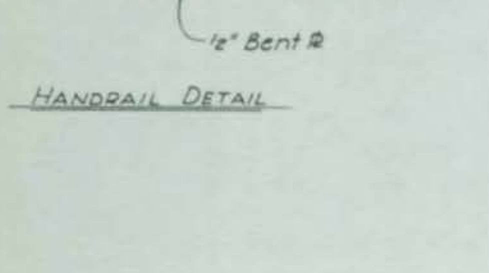
HALF SECTION NEAR PIER



HALF PLAN OF STRINGER

Showing Shear Developer Spacing

SECTION THRU STRINGER



HANDRAIL DETAIL

struts welded to the bottom flange of the outside stringer. This arrangement is not only very economical but should allow fast and economical replacement in case the rail is damaged.

Design loading on the bridge is H15-S12, which is heavier than that used on most township bridges in this area, and which should insure that the bridge will not be damaged by occasional overloading.

One question which arose in this design concerned the bearing of concrete on the narrow top edge of the web. We concluded, however, that this is not critical, since the shear connectors, acting vertically, will help transfer the slab-load to the web. In addition, the bottom layer of transverse steel will rest directly on the top of the web and help to distribute the load. There will certainly be a tendency for the slab to crack over the stringers, but, if necessary, the transverse steel can be increased to prevent this without affecting the economy of the design.

We found that the tees had sufficient stiffness to carry the dead-load in the positive moment sections without shoring; they could not, however, carry the negative moment over the piers without considerable stiffening which would involve extra steel and extra welding. We designed instead for a single bent of falsework to be placed at the center of each span to carry the dead-load of the slab during construction. Falsework piling can be easily driven at the same time as the pier piles, and the single bent of shoring will not offer a serious obstruction to the stream during construction. We believe that this will cost less than strengthening the tees enough to avoid the use of falsework.

This design was prepared at the request of John Maddox, county engineer of Stafford County, Kansas. Mr. Maddox tells us that he expects to start construction soon on the first bridge using this design, and, if the design proves satisfactory, to use it as a standard for the construction of several bridges in the county.

We do not, of course, have any actual costs, but indications are that the total cost of the bridge should run in the neighborhood of $125 or less per linear foot, or $4.00 to $5.00 per square foot, which we would consider to be quite satisfactory for this type of bridge. This compares with $150 to $175 per foot which is the usual cost for bridges designed to Secondary Highway Standards. This is by no means the final answer to the problem of low-cost bridge construction, but we do believe that it is an interesting step in that direction.

LOW-COST SCHOOL

The prime appeal of the Amite, La., Elementary School to the townspeople is a savings in taxpayers' dollars — only $7.50 per square foot. Second, is its attractive, carefully designed appearance — clean and neat, yet highly functional and adaptable.

An analysis of the most economical framing system led to the use of a rigid bent based on lightweight 12-in. steel beams using continuous design and supported by 4WF13 columns. These support a wood joist roof system. The structural steel beams and columns are exposed throughout providing the unified architectural discipline for the entire building. Only 35 tons of steel were needed; shop and field connections were all welded.

The building design is based on the separation of grades into 3 wings of 8 classrooms each, primary (1-2) in the first wing, grades 3-4 in the central wing and grades 5-6 in the last wing. If necessary, these wings can be extended to meet expansion requirements.

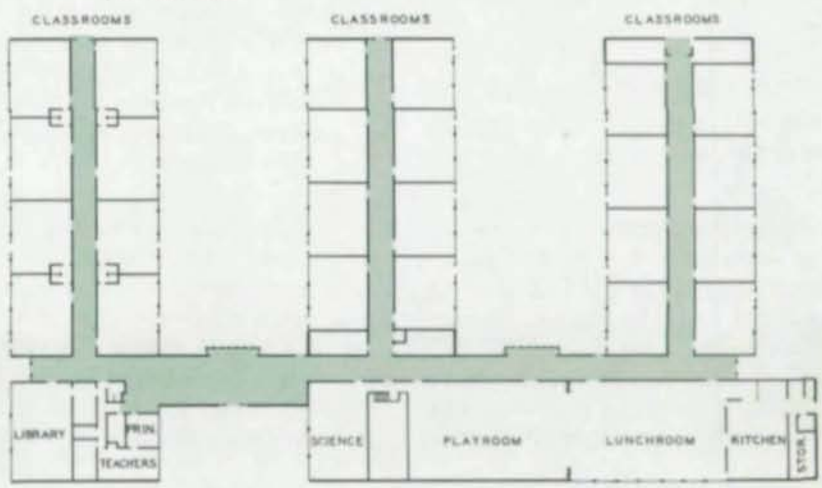
In the classrooms the floors are vinyl asbestos over concrete, the walls are corkboard and chalkboard over plywood. The kitchens have quarry tile floors and glazed tile walls.

The total cost of the entire structure was $308,085. for 40,965 sq ft.

Desmond-Miremont & Associates of Baton Rouge and Hammond, Louisiana were architects and engineers; John Belcher was the general contractor.

▲ *The classroom wings open directly to the playground at the rear of the school and to the main central corridor at the front, which in turn leads to the cafeteria, lunchroom, administration offices and busloading.*

▼ *Exterior wall materials are brick with some exposed solar tile. Interior materials are terrazzo floors and plastic walls for corridors.*



FLOOR PLAN

QUESTIONS & ANSWERS on the AISC Spec and Code

Q. The alignment chart given in the Commentary is a function of the column and girder stiffnesses, and from it, it is possible to determine the factor K to compute the effective length of the column. Is the factor K thus obtained the same for vertical loads as well as for vertical plus lateral loads?

A. The K-factor, which is used to determine allowable axial stress F_a, is the same whether the column does or does not resist concurrent bending due to lateral forces. F_a is defined in Sect. 1.6.1 as the stress that would be permitted if axial stress alone existed. Of course, as bending stress is increased due to heavier lateral loading (the vertical loads remaining the same), the importance of the term f_a/F_a, and hence the importance of K, diminishes.

Correction

A more proper answer to the question on tack welding concerning Section 1.23.6, which appeared in the first quarter issue of MODERN STEEL CONSTRUCTION, would be:

Q. Why is tack welding included in the preheating requirements in Table 1.23.6?

A. Preheating before tack welding can be considered to be at least as important as preheating before final welding, whether the tack weld is incorporated in the final weld or not. This is necessary to prevent undue hardening and possible cracking of base metal which can develop as a result of rapid cooling when there is no preheat. The cooling rate of tack welds is likely to be several times as severe as the cooling rate of normal welding.

Q. How are the provisions in the Specification and the Code of Standard Practice relating to column erection tolerances applied? These provisions are:

Spec Section 1.23.8.1: "Compression members shall not deviate from straightness by more than 1/1000 of the axial length between points which are to be laterally supported."

Code Section 7(h): "In the erection of multi-story buildings individual pieces are considered plumb, level and aligned if the error does not exceed 1:500, provided that:

(1) The displacement of the centerline of the columns adjacent to elevator shafts, from the established column line, is no more than one inch at any point in the first 20 stories. Above this level, the displacement may be increased 1/32-in. for each additional story up to a maximum of two inches.

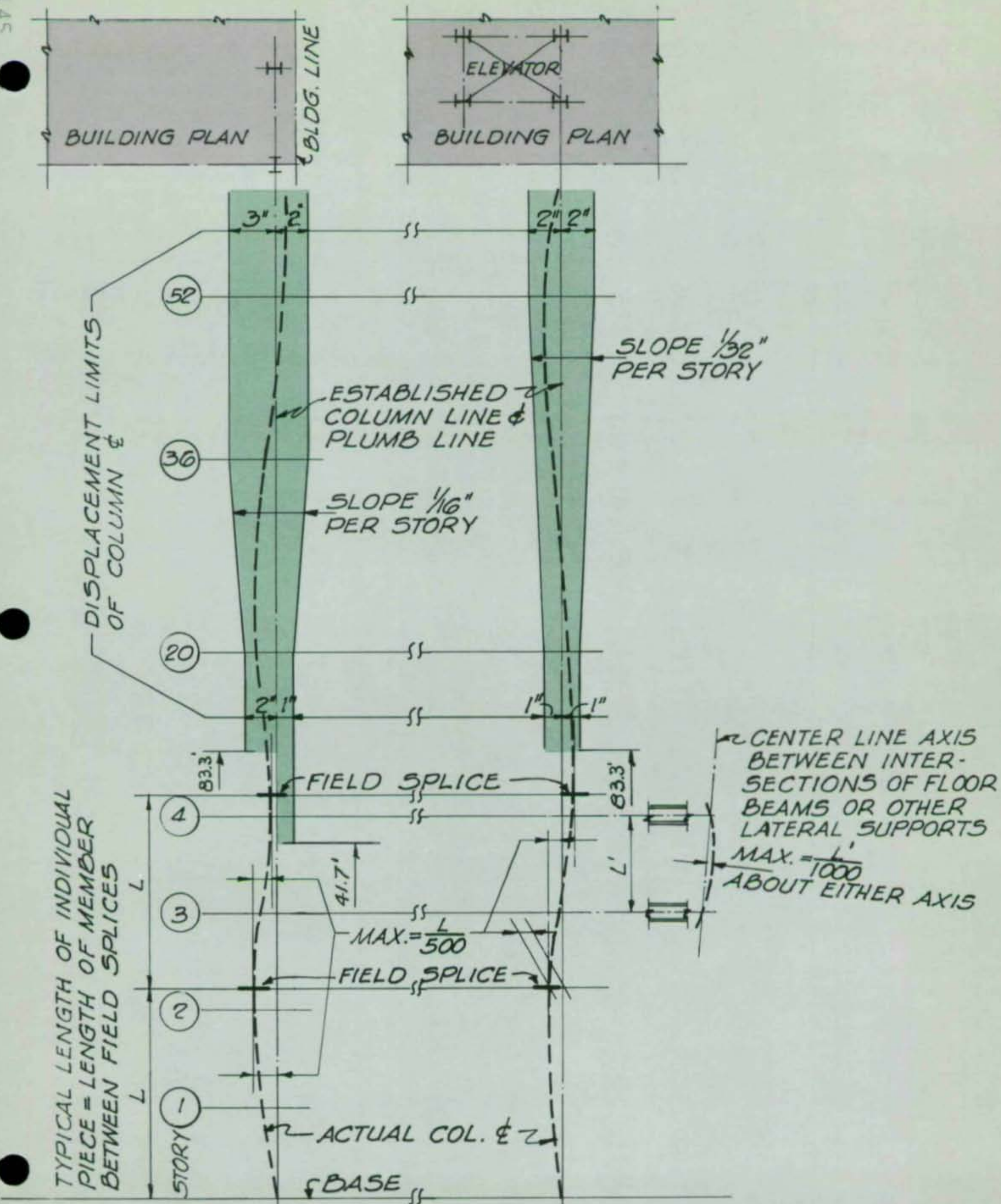
(2) The displacement of the centerline of exterior columns from the established column line is no more than one inch toward, nor two inches away from, the building line at any point in the first 20 stories. Above this level these limits may be increased 1/16-in. for each additional story, but may not exceed a total displacement of two inches toward, nor three inches away from, the building line."

A. The diagram on the opposite page shows the column erection tolerances. The envelope defined by the green area represents the maximum variation of the column center line. The broken line represents the error limitation of 1:500 described in the Code.

The reasons for allowing a smaller displacement toward the outside of the building (the building line) area are:

1. Encroachment on the adjoining property or over the property line would be more serious than having a column displaced toward the center of the building.
2. Experience by major multi-story building fabricators in New York, who formed the Subcommittee which made this code revision in 1959, indicates a tendency towards a shortening of beams in upper stories. The cumulative effect of many column-web half-thicknesses' being detailed on the "thick side" according to the Manual, as well as the effect of beam lengths being detailed 1/16-in. short, makes exterior columns tend to be displaced away from the "building line."

The limits given in the Code and Spec were agreed upon by a Subcommittee of architects, elevator manufacturers and fabricators.



ELEV. EXT. COL.

ELEV. COL.

RESEARCH CUTS STIFFENERS ON THIN-WEB PLATE GIRDERS

by William A. Milek, Research Engineer, AISC

Steel has long been a reliable and economical structural material especially adaptable and useful in the construction of bridges. It has been with us for so many years that there has been little questioning of the design concepts and criteria of its use. With the increased popularity, availability and economy of some of the newer higher-strength steels and extensive efforts to achieve maximum economy together with safety and serviceability, many of these long-accepted concepts of steel design have been brought under close scrutiny and restudy.

Perhaps the most notable development has been the highly successful plastic method of design. At the present time this concept has little applicability in the design of bridge structures; therefore it is not appropriate for discussion now. However, it has keynoted the questioning, research and restudy of many of the long-accepted design criteria by AISC. Greater emphasis has been placed on a frank recognition of proven strength as an engineering criteria. This approach is in contrast to elaborate analyses based upon criteria of incomplete strength.

A good example of the merits of proven strength may be found in the case of the thin-web plate-girder investigation which has been under way for some time. The results of this investigation have been incorporated in the new AISC Specification for buildings. Previously the simplifying assumption had been made that membrane action was inconsequential and should be ignored in the design of plate-girder webs. Extensive tests on full-sized girders proved conclusively that rather than being of minor importance, membrane action accounts for a large portion of the ultimate strength of plate-girder webs. Rather than being on the verge of collapse when the beam shear-strength has been realized and web buckling commences, a "thin" girder web accepts larger shears without distress. In the interest of economy this strength should not be ignored.

The tests were completed on static-

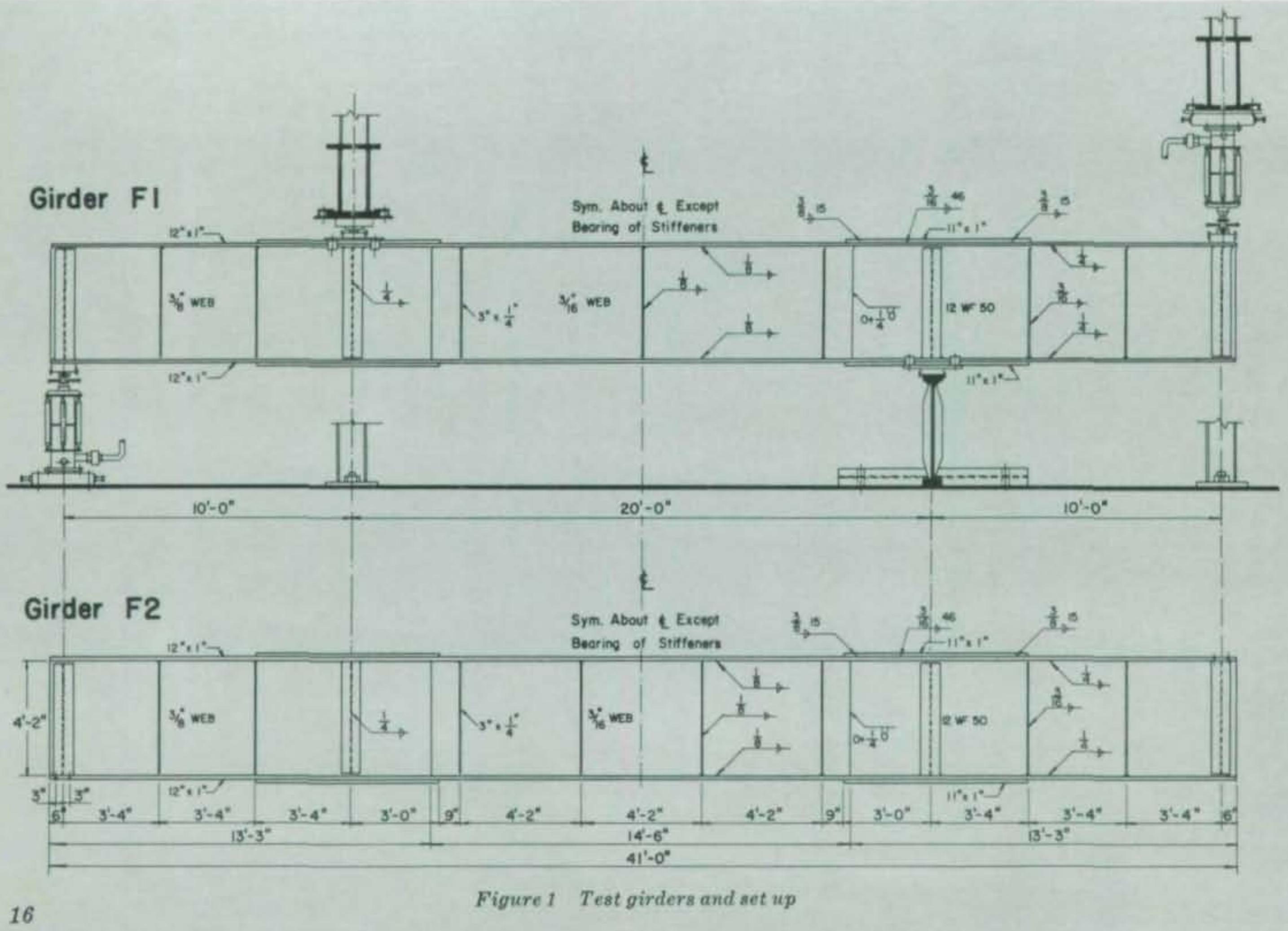


Figure 1 Test girders and set up

ally loaded girders at the time of the writing of the AISC Specification; however, it was anticipated that girders of similar proportions would prove their strength under fatigue-type loading in an equally convincing manner. The program is continuing to assemble the data to corroborate this assumption.

To date two full-scale fatigue tests have been completed and reported. The girders were 41 ft long, 50 in. deep with webs only 3/16-in. thick. These proportions reduce to a web depth-thickness ratio of 265 to 1. They differed from each other only in the spacing of intermediate transverse stiffeners employed in the test section. For the test, one girder had stiffeners spaced at intervals equal to the girder-depth. The second had stiffeners spaced at 1½ times the depth. One girder was in accordance with the provisions of the November 1961 AISC Specification, while the second was beyond the limits of the recommendations. Loading was arranged to produce high uniform shear throughout the test section. These specimens are shown in Figure 1.

The procedure used to determine the test loads is best explained by a modified Goodman diagram. As shown in Figure 2, plotting maximum loads as ordinates and minimum loads as abscissa, point A is located as the static ultimate load, that is, the load at which tension-field stress reaches yield point. Point B is located on the vertical axis indicating the maximum load when the load is cycled from zero to maximum. Its position vertically was determined by assuming the working fatigue-load to be in proportion to the ultimate load as the static working stress is to the yield point stress. Thus, $P_w = 18/33 (P_u) = 0.545 P_u$. By connecting point A to the origin with a straight line, and point A to point B with a straight line, we have a complete diagram. Load ranges from the lower to the upper line at any load-level would be expected to have equal lives. Under actual conditions, dead-load might reasonably be expected to equal about one-half the full dead plus live-load. With this line of reasoning, a range of loads from 70 to 130 percent of P_w (46.5 k to 93.0 k) was adopted for the tests. This should give the same fatigue life as that of a girder subject alternately to full static design load and no load.

It was estimated that the girders might sustain 2,000,000 cycles of loading, and in such event the range would be increased to the capacity of the testing machine after 2,000,000 cycles in order to speed up failure.

Figure 3 shows the history of the loading. The load was cycled between 46.5 k and 93.0 k at 250 cycles per minute with a shutdown for static testing and close inspection at 1,000,000 cycles. At 2,000,000 cycles the girder was static-tested to 110 k, and, upon inspection under load, two hair cracks were discovered adjacent to one stiffener. Since the cracks did not penetrate through the 3/16-in. web, an attempt was made to repair by welding over the crack without gouging. However, the cracks reappeared after 70,000 cycles. Loading continued for 500,000 cycles

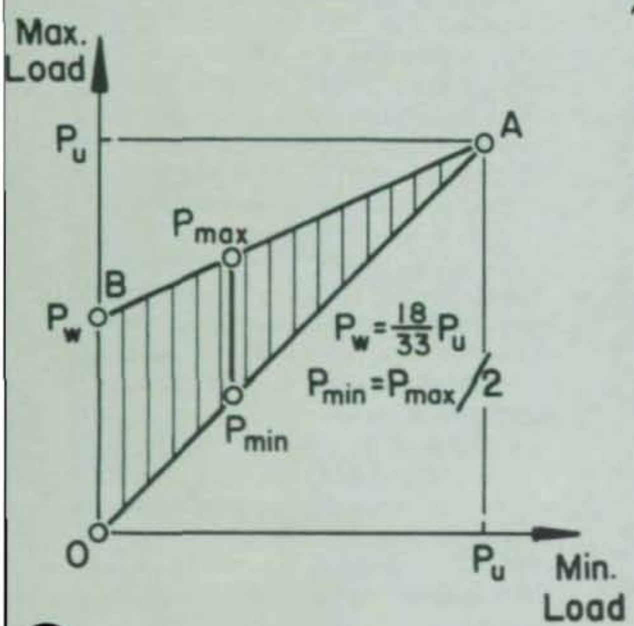


Figure 2 Test loads

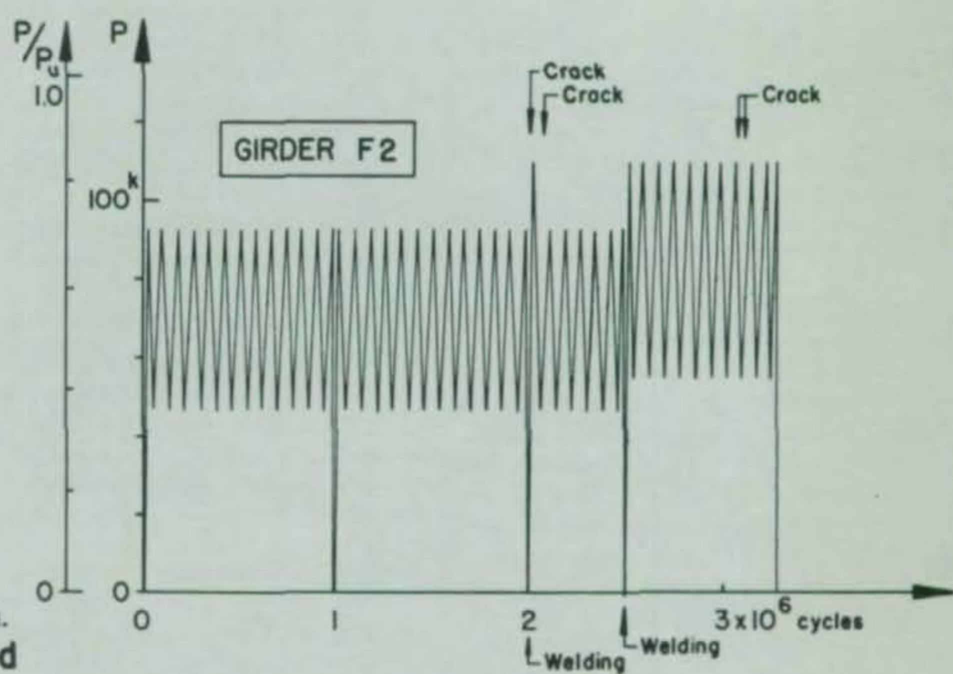


Figure 3 Sequence of testing

at which time the cracks had penetrated through the web and started to propagate upward and downward. Before too much damage had occurred the area was isolated by the welding of additional stiffeners either side of the stiffener along which the cracks had developed. The new stiffeners performed the function of taking that portion of the web and the original stiffener out of action. Now the test section consisted of two panels in which the space between stiffeners was 0.85 times the depth and one panel in which the space was equal to the depth.

Loading was resumed at a higher load range of 55 k to 110 k. After 3,080,000 cycles cracks appeared in the larger of the three panels. The cracks did not penetrate through the web and loading continued under close observation up to 3,277,000 cycles at which time the test was terminated.

As was to be expected, the second girder did not perform quite as well as the first, since it was beyond the limits of the Specification even for a static-load condition. On the other hand, with repairs made as cracks occurred, it did survive more than 4,000,000 cycles of loading which perhaps is a more convincing bit of evidence that fatigue is not a serious problem in thin-webbed plate girders.

What is the significance of these tests? Certainly, successful tests of two girders cannot be considered as conclusive, but it is important that these two girders did perform under severe fatigue-loading without exhibiting more severe fatigue problems than have existed in the past. It does demonstrate that the AISC rules for plate girders in building construction, suitably modified to the lower stress-level which bridge engineers habitually employ, may quite possibly be used in the design of plate girders for bridges.

The employment of such rules may or may not result in a great reduction in the thickness of plate-girder webs, but the elimination of some or all of the intermediate stiffeners is an opportunity for economy that is waiting to be plucked.

For example, designed in accordance with AASHO Specifications for comparable geometry and loading, the test specimen would require a transverse stiffener spacing of 17 in. in addition to a horizontal stiffener at the web thickness of 3/16-inch. If the spacing was the same as spacing used in the test girders, the web thickness would be required to be ⅜ inch. According to the new AISC rules a ⅜-in.-web would require no stiffeners at all. Thus we can safely draw the conclusion that all stiffeners can be eliminated on many plate girders with a significant cut in fabrication costs. On very deep girders, two-thirds of the stiffeners can be eliminated with perfect safety.

Further tests are planned to broaden the range of conditions tested and to provide an accumulation of data sufficient to be considered conclusive evidence of the serviceability of thinner girder webs under fatigue loading.

This article is adapted from an address delivered by Mr. Milek at the Highway Research Board in January. He describes how research has contributed significant savings by the elimination of stiffeners on plate girders.

RESEARCH PROGRESS IN BRIDGE DIAPHRAGMS

A research project to study the effectiveness of diaphragms in steel girder bridges is being conducted at North Carolina State College, Raleigh, N. C.

Objectives of the bridge project are to establish the effectiveness of diaphragms in lateral load distribution and to develop criteria for the design of diaphragms, including design data for end connections of diaphragms to stringers and efficient diaphragm spacing for load distribution.

The study, conducted by members of the Staff of the Department of Civil Engineering, is one of sixteen projects in an intensive Highway Research Program administered through the Department of Engineering Research in cooperation with the North Carolina Highway Commission and the United States Bureau of Public Roads.

The 60-ft full-scale model bridge is located on the grounds of the Highway Bridge Maintenance Department. It consists of 5—24 WF 100 stringers (designed for an overstress of 50 per cent for H-15 loading) and four sets of diaphragms of four different sizes (14WF 61, 14 WF 34, 10 WF 25, 8 [11.5) located at either quarter or third points of the span, with variable end connections from full restraint to practically hinged ends.

The Carolina Steel Corporation, in cooperation with AISC, contributed five 60-ft stringers, diaphragm material and all necessary high strength bolts for this fundamental research effort. The contribution amounted to 40 tons of steel.

Tests on the diaphragm action in the bare steel stringer grid have just been completed and data is now under analysis. In this phase of the investigation, the concrete slab was omitted on the test bridge and loading with hydraulic jacks was restricted to each individual stringer. Variables were the load position, the number and position of diaphragms, and the size of the diaphragms. Strain gages were used in the measurements of the load distribution.

A second phase of the project has begun, with the same variables, but including a three-inch concrete slab on the test bridge and provision for H-15 truckloading. No interaction between slab and stringer has been provided in order to maximize the deformations.

In the third or final phase, a standard bridge with a six-inch concrete slab in composite action with the stringers will be duplicated for experiments. To determine maximum load, the bridge will carry to a point of failure.

The bridge project is co-directed by Prof. C. R. Bramer, acting head of the Department of Civil Engineering, and M. E. Uyanik, civil engineering professor, with the assistance of civil engineering graduate students.

The aim of the Highway Research Program, coordinated by Prof. C. M. McCullough of the civil engineering faculty, is to find the most efficient and economical methods for construction and maintenance of North Carolina highways.

After all tests are completed and analyzed, a complete report on the findings and the criteria developed will be published in this magazine.

COACH HOUSE MOTOR INN

The new Coach House Motor Inn, recently completed in Milwaukee, Wis., is a single-bay, seven-story cantilevered building that demonstrates the economies which can be effected by a structural steel frame fireproofed with vermiculite plaster.

The rigid frame design reduced the framing cost about 15 per cent compared with prestressed, precast, or poured-in-place concrete. It also made a 15 per cent saving in building height.

Beams cantilevered from the columns and flange-to-flange welding achieved a closely balanced moment design for the specified floor loads and a wind load of 70 mph. The simplified full-section flange-to-flange welding detail used was considered to be 25 per cent more economical than riveting or high strength bolts for the moment connections required, and allowed half of the welding to be done in the shop.

Contact ceilings of vermiculite plaster on metal lath tied to bar joists provided the fireproofing and the finished ceilings and made a saving of 12 per cent, compared with a suspended ceiling system.

Cantilevers are expressed architecturally by curtain walls between the five upper floors and by a belt of precast units at the floor line. The column bay is emphasized by a vertical panel of precast units in which the aggregate is exposed.

The Coach House contains 102,500 sq ft, including indoor parking. Its cost was $1,350,000 or $13.17 per sq ft. The building is 77 ft high. The five upper stories are 46 ft wide and 232 ft long, with floor-to-floor spacing of ten feet. The two lower stories are 70 ft wide and 250 ft long. The first story is 15 ft high; the second, 12 ft.

The building is carried on two rows of 14-in. columns with a maximum eight-inch flange width in order to save as much floor space as possible. Columns are spaced 18 ft on center the length of the building to accommodate lower level parking. The center bay is 30 ft wide. The cantilevered sections project from the columns seven feet on one side, nine feet on the other.

The typical center beams are 16 in. deep to provide the desirable ten foot floor-to-floor height. S-6 bar joists, 12 in. deep, span the beams two feet on center. The decking is corrugated steel with a $2\frac{1}{2}$-in. concrete topping.

The cantilevered beams were shop-welded to the columns, which were fabricated in three lengths. The only bracing used were temporary cable ties and bolted erection angles. As soon as an assembly was in place on each side, the center beams were laid between and field-welded to the columns flange-to-flange. In addition to visual inspection, the welds were examined with radiographs and magnetic particle testing.

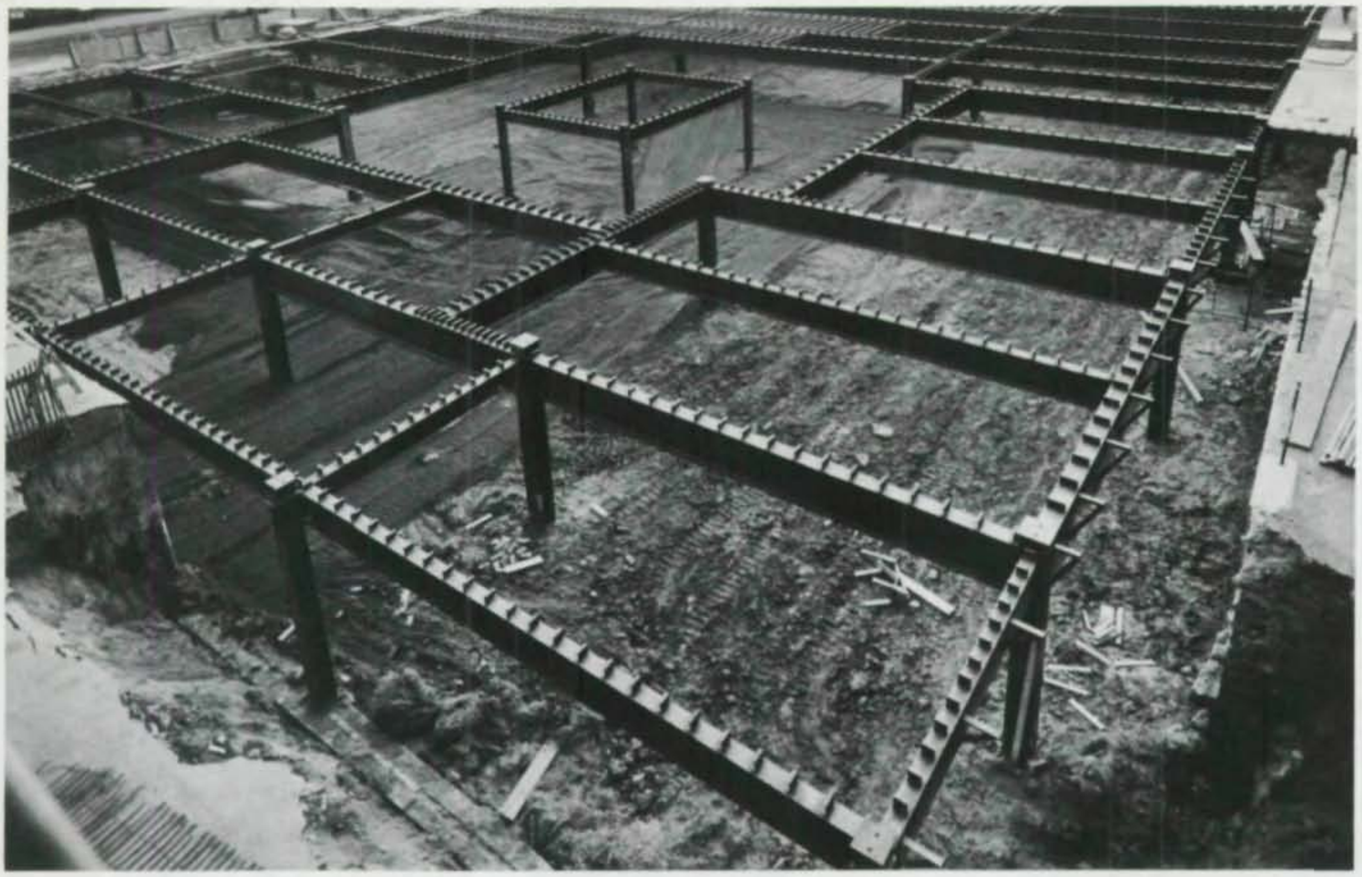
The column fireproofing is vermiculite plaster, $1\frac{3}{8}$ in. thick on self-furring metal lath. The ceiling thickness is $\frac{3}{4}$-in. on rib metal lath. Both assemblies have three-hour fire ratings.

The columns are concealed in 12-in. partitions to increase the sound-reduction value of the wall between occupancies and to keep the columns from projecting into the rooms.

Corridors are off-centered to permit rooms 19 ft deep on one side, and 20 ft-6 in. on the other, which provides a variety of accommodations in a limited space. The five upper hotel floors have 105 sleeping rooms, including five luxury suites consisting of two rooms, two baths, and an outside balcony.

The basement and the first and second floors are given over to parking space, meeting and dining rooms, kitchen and bakery, and a large banquet room. The Coach House will cater to industrial conventions that present new products, and there is an over-sized door to the second floor banquet room that permits displays seven feet high and eight feet wide to be trucked intact into the room.

Architect for the project was Sheldon Segel, AIA. The structural engineers were Collings Engineers. Steel fabrication and erection were by the Milwaukee Bridge Company, and general contractor was Drobac & Associates. All firms are in Milwaukee.



STACKING THE DECK

by Robert F. Lorenz,
Regional Engineer, AISC

The Ryan Parking Deck in St. Paul, Minn., had originally been planned as an 18-in. reinforced concrete flat slab using waffle-slab construction with 24-in. drop-panels. The scheme was rejected when the bids exceeded the project budget limitation.

James Tillit, structural engineer, with Walter H. Wheeler, architect-engineer, Minneapolis, then decided to implement the new AISC design criteria for composite design of buildings, combining a six-inch light weight concrete slab compositely with structural steel girders. Although the projected height of the building would be slightly greater with steel framing, this was minimized by use of composite action. More important, the cost was within the budget figure, and construction of the project could proceed.

The ramp, covering a full acre, consists of two levels of attendant parking and will have a capacity of 265 cars. The lower parking level is below grade, with the next level at street grade. The design includes the possibility of adding four more levels at a future date when conversion to a self-parking ramp is contemplated. The lower level is serviced by a circular ramp.

The structural steel composite girder system supporting the slab consists of two bays of 56 ft with two additional narrow bays of 39 ft and 19 ft respectively. For the typical 56-ft span, beams are composed of 33-in. shapes with 36-in. sections utilized where clearances are not critical. Beam spacing has a typical 15-ft dimension. Shear connectors are shop-installed five-inch channels placed at equal spacings along each girder's flange. A haunch varying between one inch and three inches is used to meet the sloping slab's grade requirements as well as to vary section properties when required. Standard connections are made to the columns with high strength bolts.

The structural steel is A36 material designed for 24,000 psi bending utilizing the compact section principle which AISC also introduced in the new Specification. This feature alone reduced the weight of a typical girder by as much as 1000 lbs.

Jack Geller, St. Paul, Minnesota, is the owner-contractor of the project. Structural steel was fabricated and supplied by St. Paul Foundry and Manufacturing Company, St. Paul, Minnesota.