II b

Constructions légères en éléments formés à froid
Leichtbaukonstruktionen aus kaltverformten Profilen
Light-Gage Cold-Formed Structures

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IIb

DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION

Light-Gage Steel Floor Systems Provided to Include Utilities – Proposals and Experiments

Systèmes de planchers en dalles orthotropes avec provision de contenir les installations – Propositions et expériences

Leichtstahlbleche mit Berücksichtigung der Installationen – Vorschläge und Versuche

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It is becoming increasingly necessary, when designing structures for buildings, to leave plenty of space under the floor to accomodate air-conditioning, electrical and plumbing installations. Further, reasons of economy drive designers to make the floors cooperate with the steel structures that support them, thus forming, when possible, composite systems.

Many proposals aiming to achieve one or other or even both these results have been put forward and Dr. Scalzi has mentioned them. The author of this paper has also worked on this problem and, together with Dr. Ballio, has started up a series of theoretical and experimental studies at the Polytechnic University of Milan on two types of Floor Systems that he considers of interest.

This research work, sponsored by the Italian National Research Council (CNR), concerns two types of Floor Systems. The first, the "Drawn Floor System", consists of two 1,5 mm sheets with hemispherical deep-drawings, arranged at regular intervals on a 50 cm square mesh and welded together at the contact points (figs. 1 and 2). A 4 cm layer of concrete top and bottom completes the panel, which is then ready to bear the flooring and

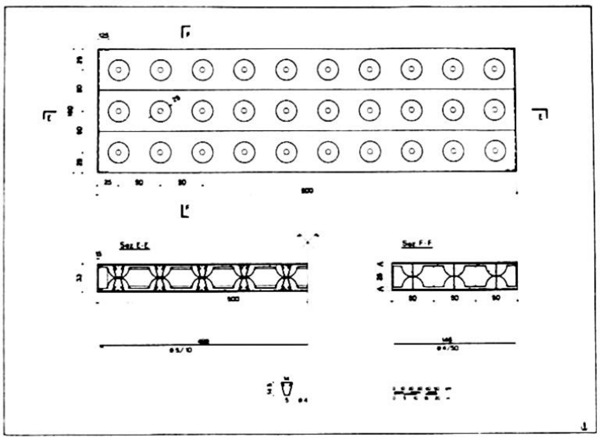

fig. 1

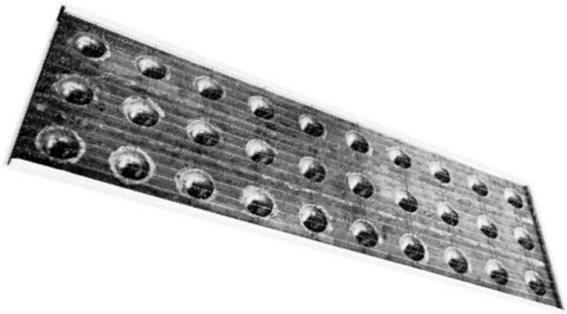

fig. 2

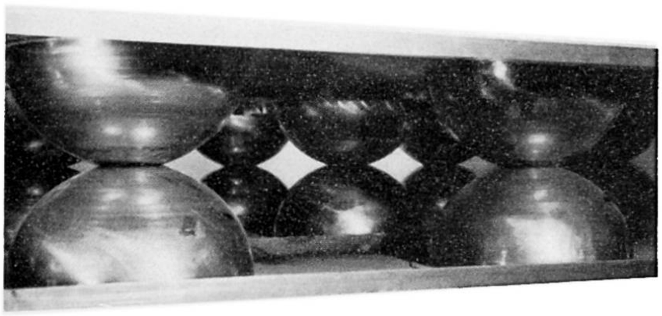

fig. 3

include the utilities (fig. 3)

No ceiling is required and the panels are automatically fireproof.

This floor system is particularly suitable because of its twodimensional plate behavior. The drawings are hemispherical simply because dies of that shape were available, but clearly other shapes (for instance, truncated pyramids) should be more suitable.

A floor system with a span of 5 m x 1,50 m has been constructed and subjected to laboratory loads tests (fig. 4).

The loads transmitted by two jacks were distributed so as to achieve as far as possible a uniformly distributed load.

The elastic line due to service loads (fig. 5) and the load-deflection diagram (fig. 6) were plotted with care.

An analysis of the experimental data supplied the behavior of the floor system, which comprises three

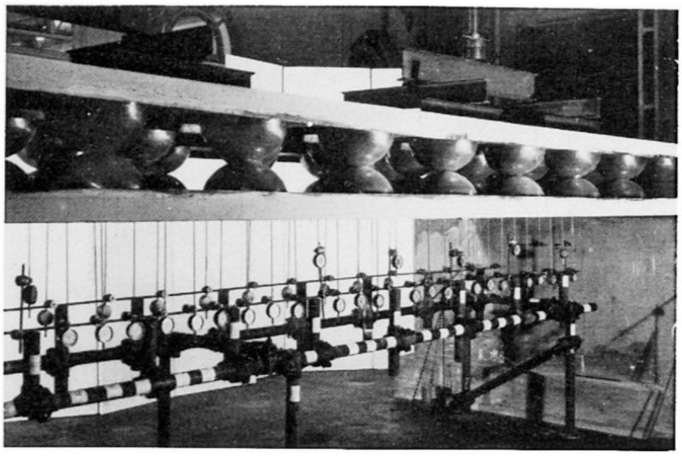

fig. 4

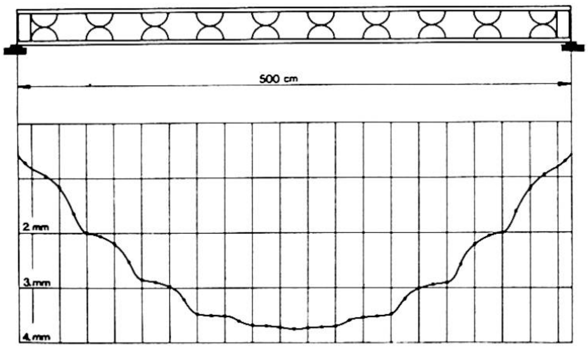

fig. 5

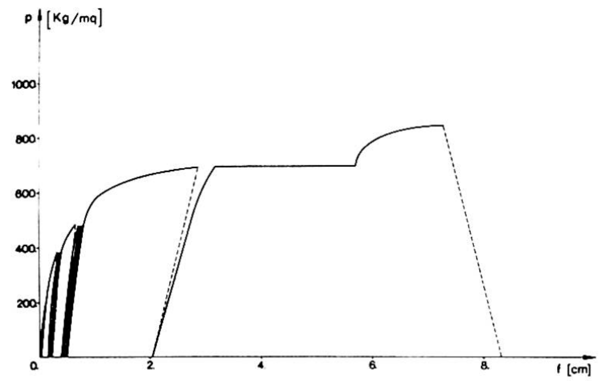

fig. 6

superposed static effects [4]:

a) flexural effect of the floor as a whole

b) shear effect of the floor

c) flexural effect of the top and bottom slabs

The incidence of the three effects may be expressed in the following percentages [5]:

$K_a = 53,5\%$ $K_b = 46,5\%$ $K_c = 0\%$

These values illustrate the importance of the shear effect of the floor. This is confirmed by the mode of failure of the floor.

Fig. 7 shows how failure occurred through piercing of the slab by the hemispherical cups in the zones near supports. Failure occurred at a load of 1.7 times the service load of 500 Kg/m².

It is important to emphasise that the behavior of the floor is of rigid plastic type: the initial deflection is little in absolute terms under service loads($f = \frac{L}{1500}$) whereas plastic adaptation under grea-

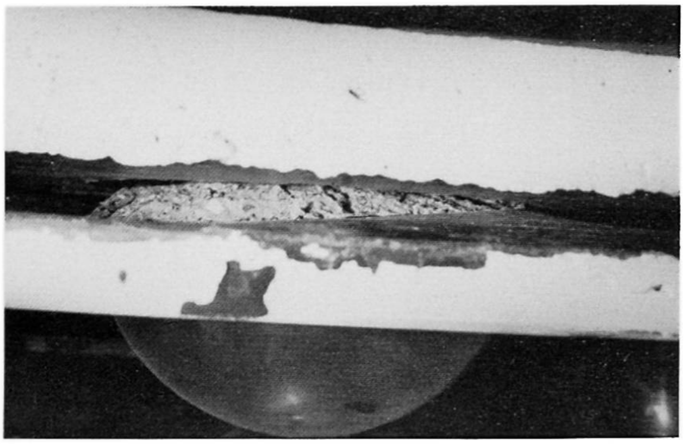

fig. 7

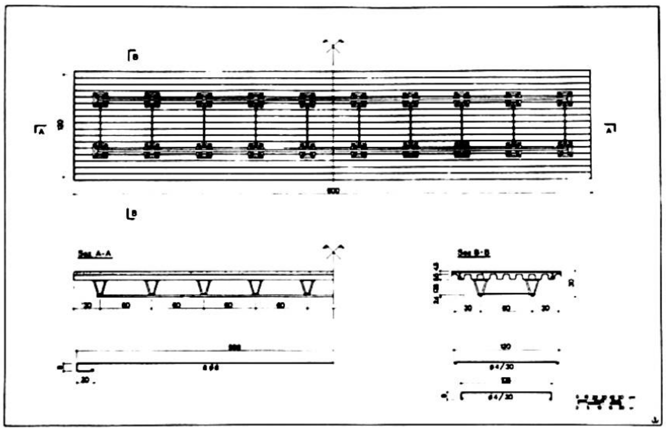

fig. 8

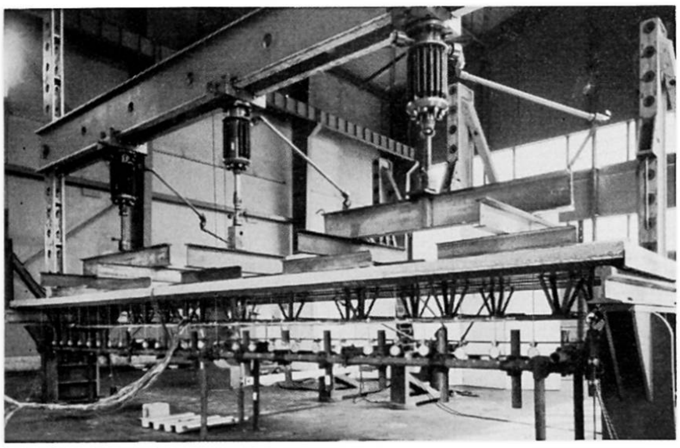

fig. 9

ter loads is considerable as proved by a ductility factor of 17. This, in the author's wiew, suggests that this type of floor system is particularly suitable for plate behavior where the redistribution of moments can favorably operate according to limit design methods.

For this reason experiments on a Drawn Floor System of 5 metres square are being arranged.

The second type of floor system that has been devised and tested (figs. 8 and 9) is an Open Type Floor System consisting of two plane parts with shear connectors between. The upper part is a cold-formed ribbed panel 1 mm thick with small drawings to act as shear connectors between the concrete casting and the steel sheet.

The total thickness of the composite slab ranges from 10 to 4.5 cm. On top there is a mesh of steel rods Ø 8 mm every 15 cm lengthwise and Ø 4 mm every 15 cm. crosswise.

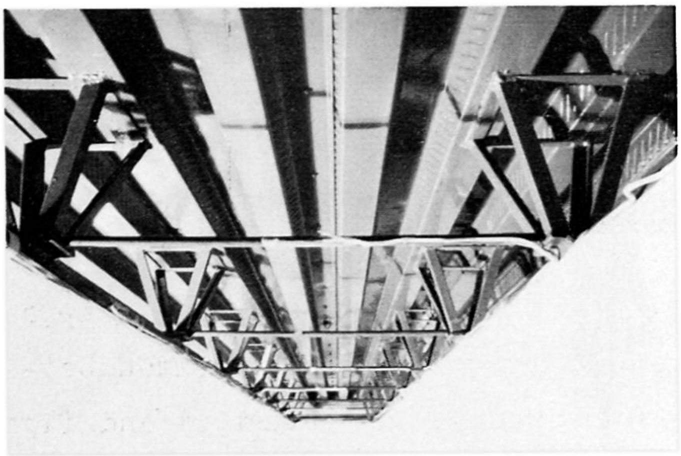

fig. 10

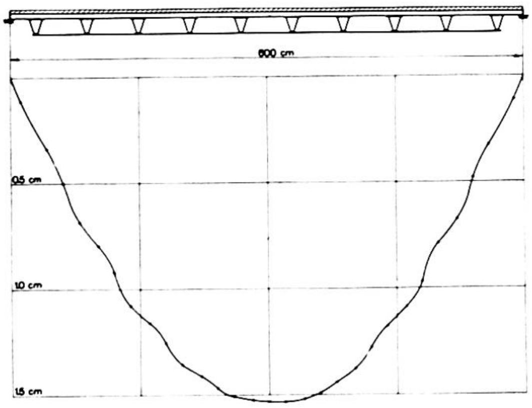

fig. 11

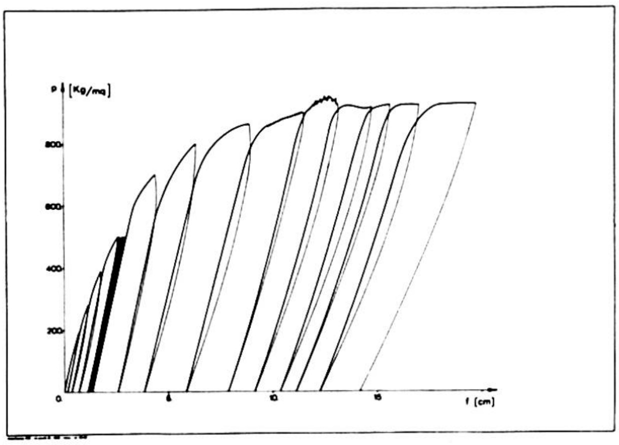

fig. 12

The lower part consists of a square mesh of steel rods Ø 24 mm lengthwise and Ø 12 mm crosswise.

The connexion elements are lattice, square-base pyramids with the apex located at the knots of the net forming the lower frame.

In this floor system, unlike the one previously described, the concrete was cast on the metal structure supported only at the ends, without need for propping at the other points.

A sample of this system with a span of 6 by 1.20 m (fig. 10) was tested. The loads applied simulated as closely as possible a uniformly distributed load.Here again the elastic line due the service loads (fig. 11) and the load-deflection diagram (fig. 12) were determined with care. The center line deflection and the load were recorded with displacement transducers and a dynamometer respectively, and plotted direct by an Hewlett Packard 7005 X,Y recorder.

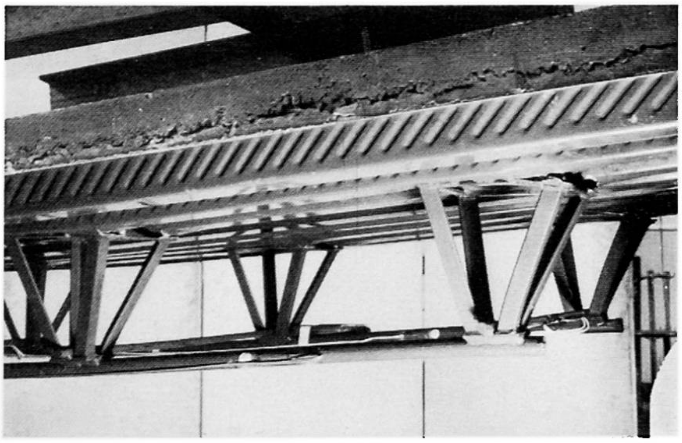

fig. 13

The behavior of the open type floor system is qualitatively similar to that of the drawn floor system but the performace is definitely superior, as is apparent from a comparison between figs. 5 and 11 and figs. 6 and 12.

In particular, the overall flexural static behavior and the effects due to shear and to upper layer work out as follows:

$$K_a = 61\% \qquad K_b = 24\% \qquad K_c = 15\%$$

Here again, the shear effect is important and its importance is confirmed by the mode of failure (fig. 13), which shows how collapse occurred through tearing of ribbed sheet away from the upper connexion of pyramids. However, the safety coefficient of the floor proved to be considerable (s = 1,95) and the deflection due to accidental service loads of 500 Kg/m^2 ($f=\frac{L}{420}$) were within quite normal limits.

For this latter floor system too the plastic adaptation was considerable: ductility factor = 7. It must be noted however that the collapse did not occur in the overall but the structure adapted herself to support a load a bit lower than the collapse one even after the crack of the connections of some pyramidal spacers from ribbed sheet. There are then good reasons for believing that also this floor system is suitable for plate behavior according to limit design methods. It must be admitted that the upper plate of mixed ribbed sheet and concrete is orthotropic and certainly more flexible and less resistant in the direction at right angles to the one tested. On the other hand, it is possible to gauge the most suitable diameter for the steel rods that constitute the bottom layer or even lay some in the diagonal direction where they would be used to best effect.

The weakest point, to wich future research must certainly be directed, is in this case too the point of connexion of the elements involved in shear effect to the slab. Before going on the further tests on floor systems it is intended to investigate this subject further, partly because on this point the problem is not only one statics but of technology and economics.

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SUMMARY

This contribution presents a study of two floor-systems using cold-rolled or cold-drawn steel sheets.

Exeperimental investigation were conducted on both floor-systems to determine the load-deflection diagram, elastic line and failure load. The results are reported and the principles of design and construction of the prototypes are outlined.

RÉSUMÉ

On présent une étude concernant deux prototypes des planchers réalisées avec tôle formée à froid. On a effectué des recherches expérimentales pour determiner la liaison entre la charge et la flèche, la ligne élastique et la charge limite.

On donne ici des renseignements sur les résultats de l'étude en soulignant les principes suivis dans le projet et la réalisation des prototypes.

ZUSAMMENFASSUNG

Dieser Bericht erläutert zwei Decken-Prototypen aus kaltgerollten oder -gereckten Stahlblechen. Experimentelle Versuche haben den Zusammenhang zwischen Belastung und Biegung einerseits sowie Biegelinie und Traglast gezeigt. Es werden die Ergebnisse, Entwurfsprinzipien sowie der Aufbau der Prototypen mitgeteilt.

Application of Light-Gage Cold-Formed Members to Modular Systems of School Construction in the United States

Application d'éléments de dalle orthotrope formés à froid à des systèmes modulaires de construction d'écoles aux Etats-Unis

Anwendung von kaltverformten Leichtbauelementen im Modularverfahren in Schulhäusern der Vereinigten Staaten

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Introduction

The nature of the building industry in the past has tended to isolate the development of building elements. The economic and technical requirements of one such element would lead to its development without full consideration being given to its integration into the total building system. There is an increasing interest in the United States in programs to reverse this trend.

One such program has been a California group formed in 1961 known as the School Construction Systems Development project (SCSD) of Palo Alto, California. This project was a joint activity of the School Planning Laboratory of the Stanford University School of Education and the Department of Architecture at the University of California at Berkeley. This activity functioned under a grant from Educational Facilities Laboratories, Inc., a non-profit corporation established by the Ford Foundation. The project architect was Ezra D. Ehrenkrantz.

A set of functional performance specifications was developed for several major components of the total building system. These specifications were used by a group of 13 California school districts for bidding purposes for establishing a modular system to be used on 22 school projects. The stated objective of the specifications was to develop "an integrated system of standard school building components which will

(1) offer architects desired design flexibility in meeting the changing program needs of individual schools,
(2) reduce the cost of school construction and give better value for the school building dollar in terms of function, environment, first cost and maintenance, and
(3) reduce the time needed to build a school."

It was felt by SCSD that often the educational methods were determined by building limitations. The specifications were formed around the premise that the building should fit the educational requirements, both present and future.

The component categories for which specifications were developed are Structure; Interior Partitions; Heating, Ventilating and Cooling; Lighting-Ceiling; Furniture; and Lockers. Each component category was required to be fully compatible and integrated with the others. All categories were to be designed to be compatible with a 5 foot by 5 foot planning module.

The Structure Category included the entire structural system except the exterior walls, the vertical shear resisting elements, the foundation, the slab on grade, and stairs. Some of the dimensional criteria were as follows:

(1) In academic areas, the maximum unobstructed area was to be approximately 7,200 square feet.

(2) The maximum depth of floor and roof construction from the top of the deck to the bottom of the ceiling was not to exceed 36 inches and would be the same depth throughout. The depth of the gymnasium roof construction would not exceed 60 inches.

(3) Changes of elevation of the slab on grade would be in increments of 2 feet.

(4) Roof spanning members were to be in 5 foot increments between 30 feet and 75 feet. Gymnasium roofs were to be 90 feet and 110 feet. Floor spanning members for use in two-story buildings were to be 30, 40, and 45 feet.

(5) Primary elements were to be available in 5 foot increments between 10 feet and 30 feet.

(6) Ceiling heights were to be in 2 foot increments between 10 feet and 18 feet. Gymnasiums were to have 25 feet from the floor to the bottom of the roof structure.

(7) Cantilevers of 5 or 10 foot spans were to be provided on roof spans.

The structural system was to be designed to the Code requirements of the Schoolhouse Section of the Office of Architecture and Construction of the State of California, which include provisions for resistance to earthquake forces. Horizontal diaphragms were to be part of the Structure Category. However, as previously mentioned, vertical shear walls or bracing were to be outside the Category. The light gage steel code requirements of this agency basically follow the provisions of the 1962 Edition of "Specification for the Design of Light Gage Cold Formed Steel Structural Members" of the American Iron and Steel Institute.

The SCSD system was bid and has now been completed with a total of 1,400,000 square feet of school buildings being built in California.

Four systems have been developed which are based on the SCSD Specifications and use elements composed of light gage cold formed steel. These systems are now described in some detail.

Inland Steel Products Company (Milwaukee, Wisconsin)

The systems developed by this company are called the Inland Modular Systems. They consist of an integrated structural system and compatible ceiling-lighting system. Both were specifically designed for and used in the SCSD program. The structural system consists of three main components: The truss-deck, the primary truss, and the column.

The truss-deck unit is a simple span truss which used a 1-1/2 inch deep light gage, cold rolled steel deck as a roof covering, as the top compression chord of the truss, and as a horizontal diaphragm. The deck flutes run parallel to the truss-deck units spaced at 5 feet on center. Light gage, cold-formed, hat-shaped purlins running perpendicular to the truss-deck units support the deck also at 5 feet on center. Vertical load supported by the deck is transferred by the purlins to the top chord panel points of the trusses. To complete the top chord connection, horizontal light gage shear plates welded to the under side of the deck distribute horizontal forces into the deck from the truss-deck web members. Thus the deck flutes act as beam-columns.

The web members are cold-formed steel tubes, 1-1/2 inches square in various thicknesses. They are welded into a U-shaped bottom chord and are cold rolled, light gage steel having a yield strength of 50,000 pounds per square inch.

Truss-deck units were designed for roof spans from 10 to 75 feet in 5 foot increments. These trusses are all 33 inches deep out-to-out. For gymnasium spans, 90, 100 and 110 foot truss-deck units having a 57 inch deep truss are used. These trusses plus most of the roof spans can support either a 5 or 10 foot cantilever.

Truss-deck units for floors were designed in spans from 10 to 45 feet.

These are similar to the roof units except that a 3 inch thick lightweight concrete slab is added. In order to achieve a composite action and transfer horizontal shear forces into the slab, Inland's Hi-bond deck was used along with a specially formed shear connector welded at the truss panel points. Inland Hi-bond deck is similar to the roof deck but with a raised pattern of slanted ridges in the vertical elements of the deck flute.

All truss-deck units are shop assembled with the deck and purlin hinged to the truss to permit folding for ease of shipment. As the deck is unfolded during erection in the field, a predetermined camber is automatically obtained. This is accomplished by horizontal camber screws pre-set in the shop which bear against the purlins.

The truss-deck units are designed for basic roof live loads of 20, 25, 30 and 40 pounds per square foot and basic floor live loads of 70 and 100 pounds per foot. In addition to supporting the normal roof and ceiling loads, all roof truss-decks have been designed to carry the roof top heating, ventilating and air conditioning (HVAC) units at any location of the roof.

Transverse distribution bracing and diagonal bracing elements were designed to serve as bridging for the truss-deck units, to collect and distribute horizontal forces and to transfer horizontal forces between the top and bottom chord levels.

The primary truss supports the truss-deck units. It is a Warren type truss with a 33 inch constant depth out-to-out. It was designed in span lengths from 10 to 30 feet in 5 foot intervals. Five and ten foot cantilevers are also available for the longer trusses. Rectangular tubular members of high strength steel are used for the top chord and web members. The bottom chord is composed of either cold rolled double channels or bars, also of high strength steel.

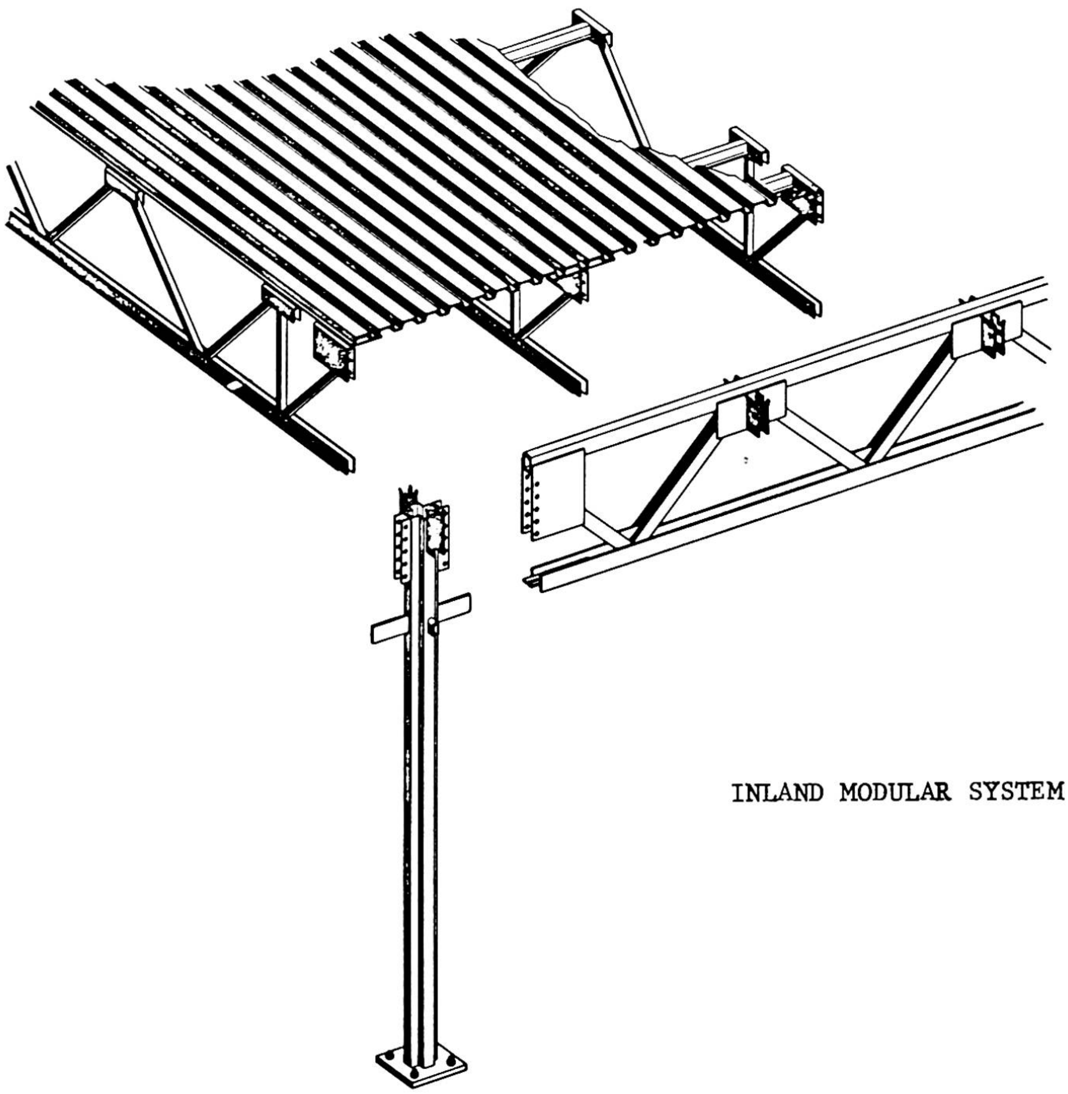
To facilitate the various combinations of truss-deck units for end and interior spans, primary trusses have been designed for a range of load capacities. Each span length is available in 9 to 10 different load classes.

Columns are hollow cruciform shaped in cross-section with a constant out-to-out dimension in each direction of 7 inches. The load-carrying capacity is varied by various wall thicknesses and by the use of high strength steels. Standard column lengths were designed in 1 foot increments to a nominal 30 feet. The cruciform shape with its indented corners has been designed for compatibility with interior partitions. It allows these partitions to intersect at columns without a corner obstruction. It also allows the standard Inland ceiling units to fit at the columns the same as they do at interior modules. One and one-half hour fire resistance rating for these columns is attained by the application of an intumescent coating at the outside and by filling the inside with concrete.

One of the principal advantages of this system, as well as all the systems described in this paper, is the speed of field erection. After columns have been placed, the primary beams are bolted to the column connectors with high strength bolts. The truss-deck units are then lifted into place between primary beams or columns. Steel pins engage in slotted end connectors. Decks are then unfolded into plate. The end connections are completed by the addition of more high strength bolts. The steel deck and purlin elements are then welded to the trusses. Fillet welds are applied between the shear plates and top gusset at each truss panel point.

Shear transfer units may be required in order to achieve lateral stability due to wind or seismic loads. Lower column connections connecting the lower chords of both the truss-deck and the primary truss offer some lateral restraint. However, since the columns are limited in their moment capacity and are primarily designed for vertical load, often additional bracing is required. This may be accomplished by fastening truss-decks or primary trusses to shear walls, by adding cross bracing between columns or by any other suitable means the engineer chooses.

After the structural system has been erected, the ceiling-lighting system may be installed. The Inland System offers metal coffers which fit into the 5 foot by 5 foot module and allow for either recessed or flush type lighting to be installed. Light gage, cold formed elements are also present in the ceiling system. The ceiling-lighting system, as well as the interior partition system, has been designed for the seismic loads required by SCSD.



INLAND MODULAR SYSTEM

The development of this system was done with thorough theoretical design and comprehensive tests. Structural tests were conducted to verify the adequacy of indeterminant welded connections, to check assumptions used to calculate deflections, and to confirm the adequacy of the criteria used in design of the entire system for sustaining the design loads. Three major tests and numerous smaller tests were conducted. An initial large scale test was performed in Los Angeles prior to the award of the SCSD contract. The purpose of the test was to demonstrate performance of the system under applied vertical, lateral, and combined loadings. The test specimen used consisted of 5 roof truss-deck units forming a 25 foot by 55 foot bay. The test demonstrated close conformance with predicted stresses, deflections, and determined diaphragm shear values.

A floor test was then performed using a 10 foot by 45 foot specimen. Loading was applied in three phases over a 6 month period. In addition to measuring deflections and the amount of rebound after initial and final loadings, creep during a 6 month duration was measured under static load. Test cylinders determined concrete strength over this same time period. Composite action between the steel deck and concrete slab was confirmed, as well as the adequacy of the special shear connector and the ability of the truss-deck units to sustain more than twice design loads.

A third major structural test was performed using a 20 foot by 70 foot roof truss-deck specimen. The major objectives were (1) to determine the performance of the purlins to both support the deck and provide bracing for this compression member, (2) to examine the stresses around framed openings in the deck, and (3) to measure stress distribution across the deck between trusses. Several of the purlins were bridged across before loading was applied in order to isolate the stresses due to staying the compression member from the direct vertical load stresses. Strain gages were applied at critical areas. Test results indicated (1) bracing action exhibited by the purlins was less than 1% of the deck compressive force, (2) the reinforcement used around large openings was more than adequate, and (3) the stress distribution in the deck was concentrated at the panel points at purlins and distributed between purlins confirming the theoretical analysis.

Fire tests were also performed under the auspices of the Underwriters Laboratory in Chicago, Illinois. Ceiling-lighting components, columns, roof and floor systems combined with the ceiling-lighting system were among those items tested. As a result, a 1-1/2-hour rating for the columns, and a 1-hour rating for the roof and ceiling assembly was obtained.

Butler Manufacturing Company (Kansas City, Missouri)

The system developed by this company is called the Space Grid System. It was developed for bidding the SCSD system. However, the floor elements are not now included as part of the system.

The horizontal spanning members of this system are simple span pyramidal shaped Space Trusses assembled adjacent to each other and bolted together. The Space Trusses maintain the basic 5 foot by 5 foot module. All members of the Space Truss are light gage cold-formed steel shapes. The lower chord of each Space Truss is composed of two approximately 5 foot on center shapes. When each is bolted to its mate of an adjacent unit, a 1 inch space is maintained for passage of air, electrical, and water services between the roof-ceiling envelope and the room space. A fluted transverse base member fastens each lower chord shape of the Space Truss element at 5 foot intervals. Ceiling and lighting panels are then supported by the lower chords and the transverse base members.

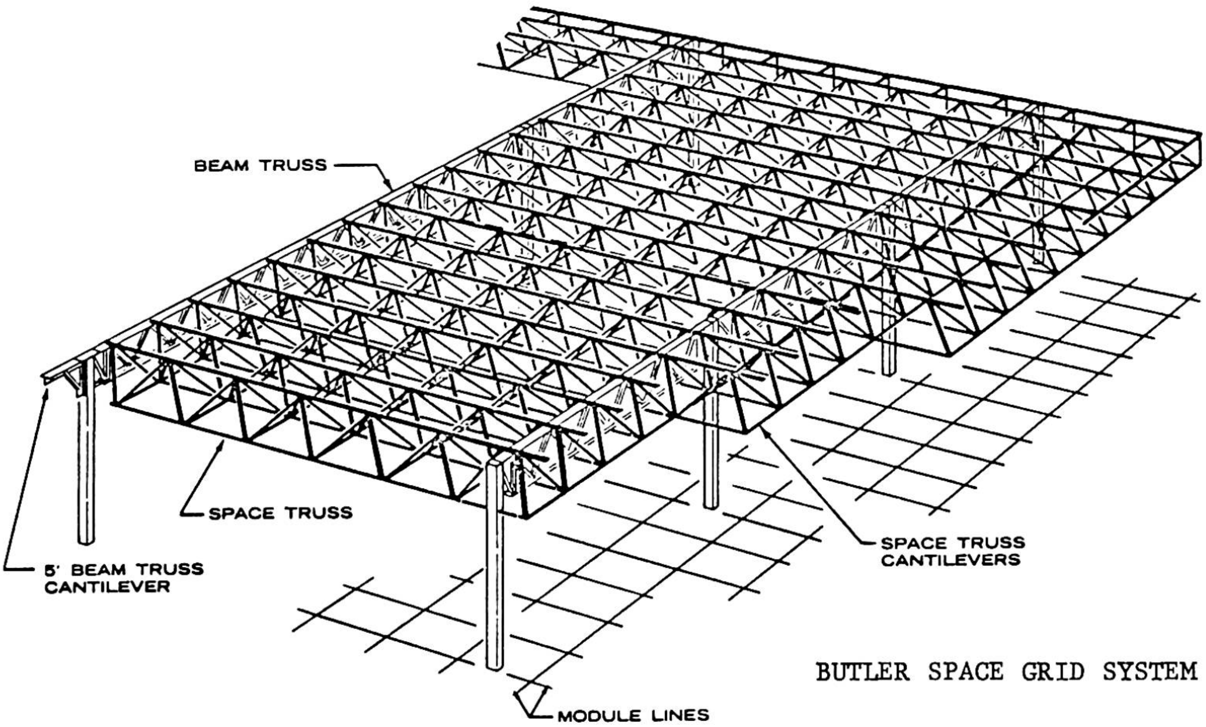
The upper chord is a fluted shape set midway between the two lower chords. Web members are bent plates welded and bolted to the upper and lower chords in the form of a pyramid. The Space Trusses are 35" deep out-to-out.

The Space Trusses are available in 5 foot increments from 20 foot spans to 40 foot spans. Cantilevers of 2-1/2, 5, 7-1/2, 10, and 12-1/2 feet are also used. The design of the Space Trusses and their supporting elements are based on three basic loading requirements as follows: (special load categories can be designed)

(1) 20 pounds per square foot live load or 15 pounds per square foot wind load with a maximum Space Truss span of 40 feet.
(2) 30 pounds per square foot live load plus 20 pounds per square foot wind load with a maximum Space Truss span of 30 feet.
(3) 40 pounds per square foot live load plus 20 pounds per square foot wind load with a maximum space truss span of 30 feet.

The Space Trusses are supported on tapered steel beams or Beam Trusses composed of welded structural tees and angles. The beams or Beam Trusses are available in 5 foot increments for spans between 30 feet and 75 feet at a depth of 31 inches out-to-out. For gymnasium spans, the beam or beam trusses are deeper than the standard envelope. Cantilevers are available in 5 and 10 foot spans. Mechanical passage is achieved through the Beam Truss or at the tapered ends of the beams. Specially reinforced holes through the beam web can also be used. One or two-hour ratings can be obtained using compatible fire-rated elements.

The beams or Beam Trusses are supported on 8-inch square tubes. Columns are of lengths to provide 9, 10, 11, 12, 14, and 16 foot ceiling heights. Other heights and changes of level are available within certain limitations.



BUTLER SPACE GRID SYSTEM

The roof covering is normally provided with an 1-1/2 inch deep light gage cold rolled steel deck which in combination with the Space Truss system acts as a diaphragm for resisting horizontal forces due to wind or earthquake.

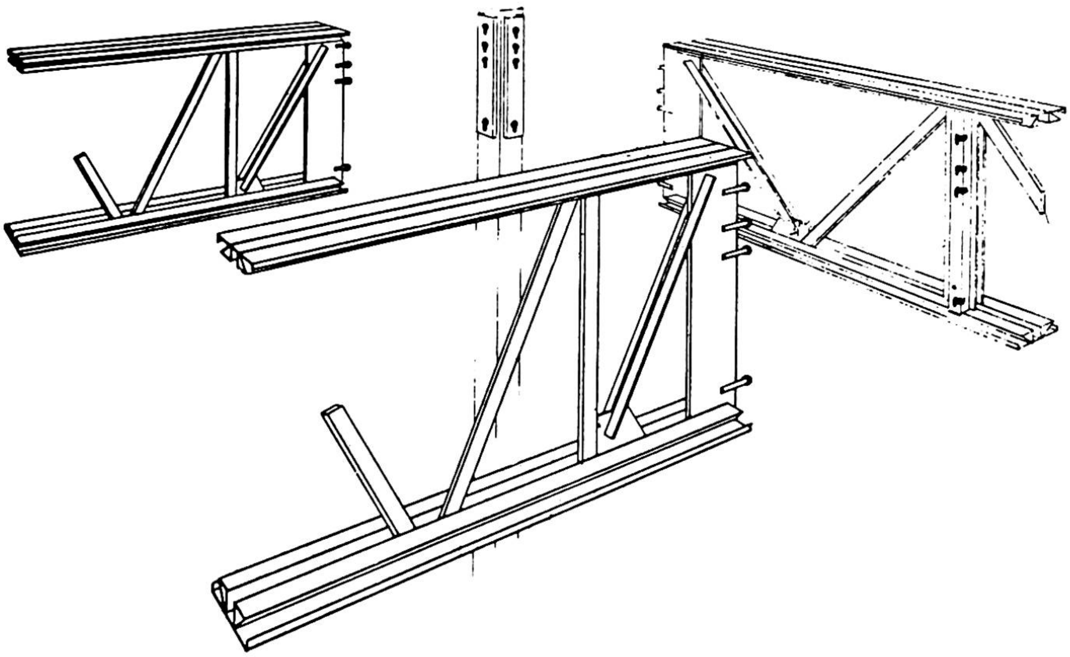
There are several fascia configurations offered in light gage steel. However, the Space Grid system is adaptable to many different fascias as may be required by the architect.

Macomber Incorporated (Canton, Ohio)

The system by this company is called the Macomber V-Lok Modular Component System (VLMC). It was developed subsequent to the bidding on the SCSD Specifications but the design principles of the system would permit it to be applied to the SCSD requirements. The columns, which would be located on the 5 foot by 5 foot planning module, can be either cold rolled or hot rolled tube sections or wide flange structural shapes. The standard column sizes are 5 inches by 5 inches, 6 inches by 6 inches, or 8 inches by 8 inches. Within these limits, the engineer can select the column required by his design from three cold-formed tube columns provided by Macomber, or any size listed in the "Manual of Steel Construction", Sixth Edition, of the American Institute of Steel Construction. Standard ceiling heights are 9 feet and 10 feet but may vary at the option of the architect or to meet partition disciplines.

The open web girders are supported by the columns and are connected to them with a special interlocking device called the V-Lok connection. The chords are cold rolled steel shapes. The webs are of tubular elements. The out-to-out depth of the girders is a constant 36 inches. Five and ten foot cantilevers are available for most girder back-up spans. The girder spans are in 5 foot increments up to a maximum of 45 feet. The girders are designed for 7 load classes, depending on the applied panel point loading.

The open web purlins are formed using cold rolled chords and tubular webs. The purlins are connected to girders and columns using a V-Lok connection similar to that used to support the girders. The out-to-out depth of the purlins is 36 inches for roof spans to 80 feet and floor spans to 50 feet. For long span roofs from 80 to 110 feet, the depth is 60 inches. Most purlins can support 5 or 10 foot cantilevers. Roof purlins are designed in 5 load classes of live load between 20 and 50 pounds per square foot. Floor purlins are designed in 6 load classes of live load between 40 and 100 pounds per square foot. Most purlins have also been designed to support the loads from mechanical unit components. All roof purlins are cambered for the maximum total deflection occurring with the member fully loaded. Floor purlins are cambered for dead load deflection.



MACOMBER V-LOK MODULAR COMPONENT SYSTEM

The roof covering is achieved using a 2 foot wide, 1-1/2-inch deep, cold rolled steel deck. The steel deck serves as a horizontal diaphragm to resist wind or earthquake forces.

Some sub-assembly elements have been provided, such as reinforcements around floor or roof openings, lower chord and lateral force bracing elements, fascia attachment elements, and wall lateral support elements.

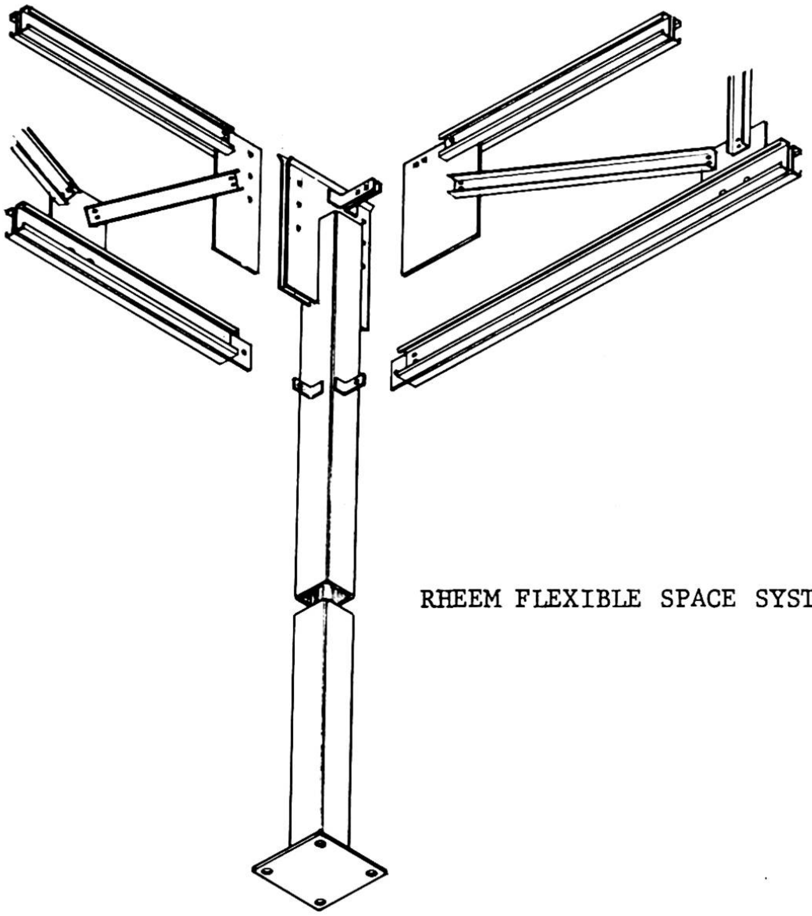
Combinations of compatible integrated components can be arranged to achieve a fire rating of up to 2 hours.

Rheem/Dudley Buildings (Paramount, California)

The system marketed by Rheem/Dudley Buildings is called Rheem Flexible Space Systems. It was developed by Compatible Design Systems of Santa Clara, California, subsequent to the awarding of the SCSD bid. The design principles of the system were based on the SCSD Specifications including compatibility with the various component categories. The main components of the structural system are framing trusses, primary trusses, columns, and roof deck.

The framing trusses are on the standard 5 foot module. The framing trusses of the Pratt type are available in roof spans at increments of 5 feet up to 75 feet with an out-to-out depth of 34 inches. Floor framing trusses vary in 5 foot increments to a span of 50 feet providing elements for 2 or 3 story buildings. For longer spans, the trusses are 64 inches out-to-out and are in 10 foot increments to 120 feet. Five and 10 foot cantilevers are also available for floor trusses and both depths of roof trusses. The chords of the trusses are two 4-inch channels ( ][ ) with varying gages for the various spans. Web members are two 2 and 3-1/2-inch channels ( ][ ) typically made with 14 gage steel. Webs and chords are bolted to 3/16-inch gusset plates. The top chords are sloped

1/4-inch per foot for drainage. At present, all truss joints are fastened using 1/2-inch high strength bolts.



RHEEM FLEXIBLE SPACE SYSTEM

The framing trusses are bolted to columns or primary trusses. The primary trusses of the Warren type are in 5 foot increments from 10 foot spans to 40 foot spans. Five and 10 foot cantilevers are also available. The chords are composed of 6-inch, light gage cold rolled channels (⊐⊏) or hot rolled channels (]\[ ). The webs are composed of 3, 4, or 5 inch hot or cold rolled channels of varying depths and gages. Webs and chords at present are bolted to 3/8-inch gusset plates with 3/4 inch high strength bolts. Primary trusses were designed in 9 load classes to accomodate interior and exterior conditions with economy.

The primary trusses and framing trusses are high strength bolted to a connector plate mounted on 6-inch square tubular columns of varying wall thicknesses. Column lengths, which can vary in one-foot increments up to a maximum of 40 feet, are determined according to the nominal roof elevation at the column. Columns are designed as pin-ended members and are divided into five different load classes.

The floor and roof decks are 1-1/2-inch fluted steel decks spanning perpendicular to the framing trusses with gage determined by the vertical loading or by the horizontal loading with the deck acting as a horizontal diaphragm. The deck is fastened to the framing trusses at predetermined points by welding or mechanical fasteners as required. On floor decks a 2-1/2 inch poured-in-place lightweight concrete fill is used.

Horizontal distribution bracing and diagonal struts are used between framing trusses to act as bridging and to distribute horizontal forces to the diaphragm.

The ceiling is directly supported by framing and primary trusses. A fascia may be supported on perimeter trusses.

In excess of the SCSD requirements, a compatible cross-brace assembly is provided to act as a shear wall on the primary truss lines if it is required.

All roof systems are designed to support the associated dead loads including mechanical equipment and live loads of 16, 20, 25, 30, and 40 pounds per square foot. Floor system live loads are a basic 70 or 100 pounds per square foot. One-hour ratings on columns and roof system have been given by code authorities.

All elements of the structural system were designed using the standard design criteria. No special testing has been required.

SUMMARY

The SCSD approach to the school building construction has aroused a nationwide interest among manufacturers, architects, and engineers in the use of integrated systems for various types of building construction. The systems developed under these specifications have been used successfully not only for schools but also for many industrial and commercial buildings throughout the United States. It has not only resulted in a more functional utilization of materials, but has provided a means of creating more efficient structures at reduced cost. New programs patterned after the SCSD program are presently under development and are evidence of the success of the concept of the SCSD System's approach to building construction.

RÉSUMÉ

La méthode SCSD pour la construction d'écoles a soulevé de l'intérêt dans tous les Etats-Unis chez les fabricants, les architectes et les ingénieurs pour l'utilisation de systèmes intégrés pour différents types de construction. Les systèmes développés sous ces spécifications ont été employés avec succès non seulement pour des écoles mais aussi pour bien des constructions industrielles ou commerciales dans tous les Etats-Unis. Il en a résulté non seulement une exploitation plus fonctionnelle des matériaux, mais aussi un moyen de projeter des structures plus efficaces à des prix réduits. De nouveaux programmes sortis du programme SCSD sont actuellement en cours de développement et démontrent le succès du concept de l'SCSD.

ZUSAMMENFASSUNG

Das SCSD-Verfahren hat für den Schulhausbau ein weites nationales Interesse unter Herstellern, Architekten und Ingenieuren hervorgerufen, wenn es sich um integrierte Systeme verschiedener Bauweisen von Gebäuden handelt. Dieses unter den beschriebenen Vereinfachungen entwickelte System ist nicht nur bei Schulen, sondern auch für viele Industrie- und Geschäftsbauten überall in den Vereinigten Staaten angewandt worden. Dieses Verfahren ist nicht nur Ergebnis einer besseren funktionellen Materialanwendung, sondern folgte auch aus der Absicht wirtschaftlicher Tragwerke und verminderter Kosten. Neue aus dem SCSD-Programm hervorgegangene Verfahren stehen zurzeit in der Entwicklung und sind Beweis des Erfolgs obigen Systems.

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IIb

Floor Systems with Composite Form-Reinforced Concrete Slabs

Systèmes de planchers en profilés de béton armé renforcés d'acier en action combinée

Deckentragwerke (Leichtbleche) im Verbund mit Stahlbetonplatten

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INTRODUCTION

Reference is made by Dr. George Winter[1] and Dr. John B. Scalzi[2] to the development and use in the United States of many variations of light-gage steel panels which may serve as roof decks, floor decks, and walls. Particularly, Dr. Winter mentions the use of a floor slab system involving concrete permanently placed over a light gage steel panel. In this case, the steel panel may perform the dual role of functioning as a form for the concrete at the construction stage, and as positive moment reinforcement for the slab under service conditions. Thus, there may be composite action between the steel panel and the concrete. Dr. Winter further points out that it is possible to utilize composite action between the slab and the supporting beams or girders.

There are many advantages in using floor systems which employ light-gage steel panels to act in composite fashion with concrete. Obviously, eliminating the necessity of installing and removing wood forms can be cost-saving, particularly in cases where the contractor cannot take advantage of form reuse. Secondly, the light gage material is easily handled and placed, hence rapid construction is possible with a minimum of on-site labor. A third advantage is that several manufacturers have developed pre-engineered raceways for electrification, communication, and air distribution which can often be economically blended with their respective systems.

This discussion describes the use of form-reinforced concrete slabs in buildings. It presents the current state-of-the-art, with emphasis on the methods of obtaining composite action between the concrete and the light gage steel, as well as between slab and supporting members. Design concepts pertaining to both types of composite action are reviewed. The last section contains photographs which illustrate some interesting applications.

COMPOSITE FORM-REINFORCED CONCRETE SLABS

Light gage steel forms for composite form-reinforced concrete slabs are commercially available in a variety of shapes and sizes. Form units normally are corrugated to provide adequate bending strength and have some type of corrosion-resistant coating such as galvanizing. A typical unit might be 2 ft wide, 15 ft or more in length and weigh approximately 2 lbs per square foot.

The thickness of the sheet ranges from approximately 24 gage (0.024 in.) to 14 gage (0.075 in.).

The forms can be separated into two basic categories based on their means of developing shear resistance. Category I (Fig. 1) is the type of form which develops shear resistance primarily through the medium of wires welded to the top surface of the form[3]. Thus, the wires become embedded in the concrete and transfer the horizontal shear into the form at the points of weld. Forms of Category II (Figs. 2 and 3) have indentations or embossments which are rolled into the material in such a way as to provide shear resistance and vertical interlocking between concrete and steel[4,5]. Actually this type of form depends to a great extent on its transverse bending strength for much of its capacity to develop shear resistance.

The design principles for form-reinforced concrete slabs are based on those pertaining to conventional reinforced concrete design[6]. Design is based on allowable valves of concrete stress, steel stress, and shear transfer. It is assumed that concrete cannot withstand tension, i.e. the section is cracked to the neutral axis; and transformed sections are calculated accordingly. The sectional properties of light-gage steel forms are computed on the basis of commonly accepted procedures[7], and each supplier provides this information for his own product. The supplier also provides other design data, such as load tables, which would pertain to a variety of conditions.

The determination of bending stresses in any composite form-reinforced concrete slab is based on the well-known flexure formulas

$$f_s = \frac{M}{S_b} \tag{1a}$$

or

$$f_c = \frac{M}{S_t} \tag{1b}$$

where

M = the applied bending moment

f_s = the stress in the bottom fiber of the steel form

f_c = the stress in the top fiber of the concrete slab

S_b = the section modulus of transformed section, bottom fiber

S_t = the section modulus of transformed section, top fiber.

The determination of shear transfer stresses can best be discussed by considering, separately, the two categories of forms. For the forms of Category I, a relationship for the spacing of the welded transverse wires is found from the formula

$$v = \frac{V}{bjd} \tag{2}$$

where

v = is the horizontal shearing unit stress in the slab between the neutral axis and the level of the steel

b = the width of slab under consideration

V = the external shear force acting

j = ratio which defines arm of resisting couple

d = distance from top of slab to centroid of form steel.

Since a steel form with transverse welded wires was used, Eq. (2) must be modified. Let

S = the spacing of transverse wires

W' = maximum allowable weld shear per wire weld

g = transverse width of repeating section assuming one weld within each section.

It follows then, by applying Eq. (2) to an area of slab S in. long and g in. wide, that

$$W' = \frac{VSg}{bjd} \ . \tag{3}$$

Equations (1) and (3) provide the means of determining required steel area and transverse wire spacing for a simply supported one-way slab, based on the load carried by the composite section. Naturally, other design considerations, such as form deflection under the dead weight of wet concrete must also be taken into account.

Shear transfer for forms of Category II is treated in essentially the same way as described above. There are two cases to consider, however, which are denoted as Category II(a), and Category II(b). Category II(a) is the case where the light gage section has embossments which are primarily on a horizontal surface at one discrete interface (see Fig. 2 and Refs. 4 and 8). In this case, the design is based on the relationship

$$t = \frac{VQ}{I} \tag{4}$$

where

t = the shear transfer force, per unit length, at the level of the horizontal interface under consideration

Q = the statical moment

I = the moment of inertia of transformed section.

In the case of forms of Category II(b), where embossments are arranged on inclined surfaces, the following relationship is used.

$$u = \frac{V}{\Sigma_o jd}$$

where

u = the average unit bond stress on contact surface between steel and concrete

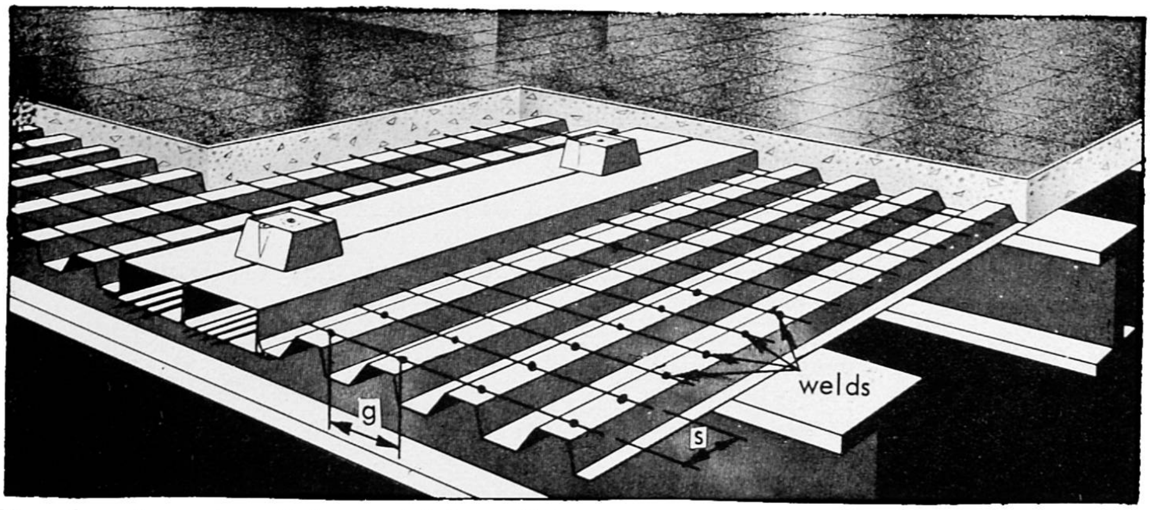

Fig. 1. Example of form which utilizes transverse wires (Category I).

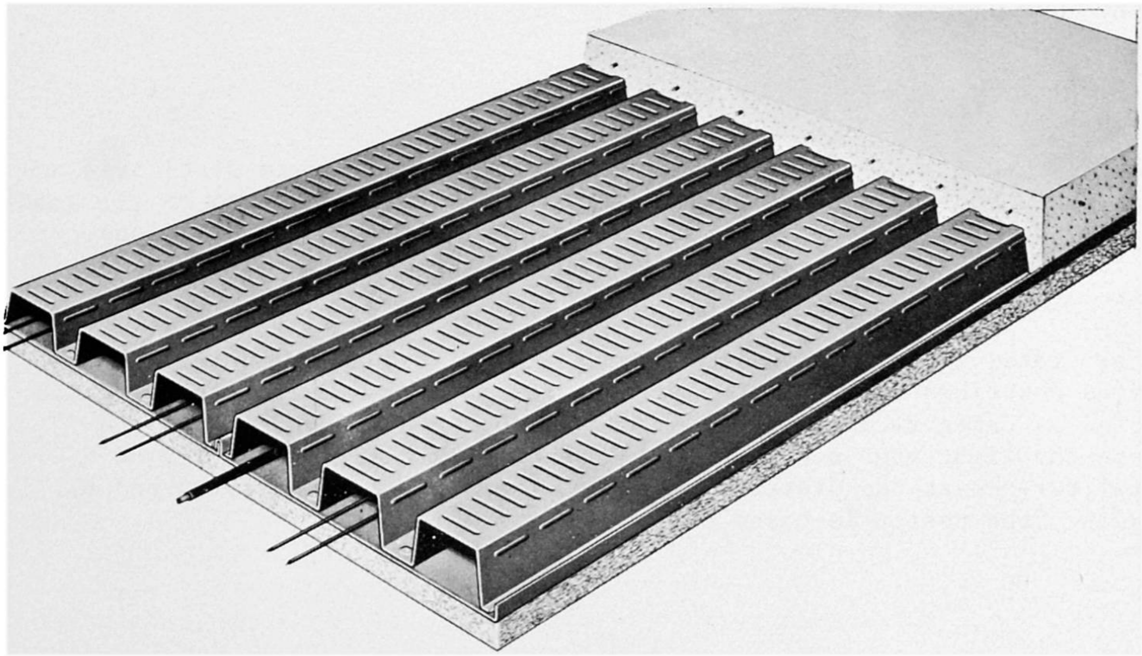

Fig. 2. Example of form which utilizes embossments on flanges and webs (Category II[a]).

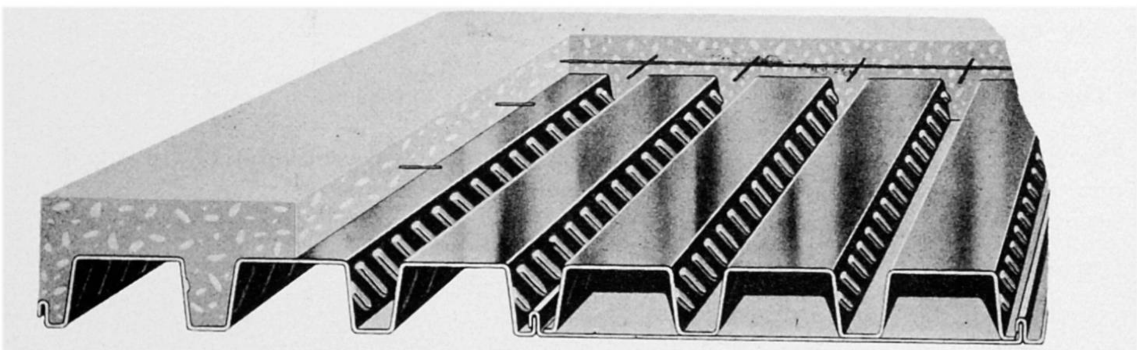

Fig. 3. Example of form which utilizes embossments on webs (Category II[b]).

Σ_o = the contact surface per unit of length.

The above is illustrated in Fig. 3, and design data given in Ref. 5.

The current state of development of form-reinforced composite concrete slabs is the result of a somewhat independent effort by steel producers. A detailed examination of the separate design criteria by these firms does generally reveal employment of sound engineering principles. Further research is needed, however, which will lead to the development of commonly accepted specifications. The co-authors of this discussion are presently engaged in such a research program at Iowa State University under the sponsorship of the American Iron and Steel Institute. The primary objective is to obtain information on the application, use, and design of concrete slabs with composite steel forms which might lead to design specifications. In view of the fact that this type of construction is relatively recent, and new shapes and configurations of light-gage steel forms are anticipated, one phase of the Iowa State research has dealt with the problem of form evaluation. The emphasis has centered on evaluating the shear transfer characteristics of the various kinds of forms. Due to the fact that the research is still in progress it is not possible to report any results at this time.

COMPOSITE BEAMS WITH FORM-REINFORCED CONCRETE SLABS

The establishment of composite action between the light gage steel form and concrete leads to the possibility that the form-reinforced floor slab may be designed to act compositely with supporting beams and girders. It must be recognized, however, that current knowledge should be strengthened to cover and guide the complete design of such systems. This is because information covering the design and construction of composite steel beam floor systems is based upon research work involving conventionally reinforced concrete slabs anchored to the top flange elements by some type of mechanical shear connector[9,10].

The performance of composite beams utilizing form-reinforced concrete decks is primarily dependent upon 1) the type of mechanical shear connectors and 2) the geometry of the steel forms[11]. A number of steel form suppliers have conducted individual research concerning this type of composite system. Design data is usually provided by these firms and in all cases applies only uniquely to their product. For example, Fig. 4(a) shows a typical composite section beam detail with a newly developed mechanical shear connector shown in Fig. 4(b). These shear connectors are welded through the steel form to the top flange elements. Figure 5 illustrates a typical composite section girder detail. Most popular are stud shear connectors. A typical composite section beam detail, employing this type of connector is shown in Fig. 6. There are two means of welding the connectors. In Fig. 6(a), the connector is welded through the steel form. In Fig. 6(b), the connectors are welded in an open space between the ends of the form. The latter procedure is primarily due to the designer's concern that chemical coatings, such as galvanizing, may hinder complete fusion in the welding process.

It is evident that additional investigation of individual floor systems is necessary to develop information leading to specifications governing composite steel beams supporting, form-reinforced concrete decks.

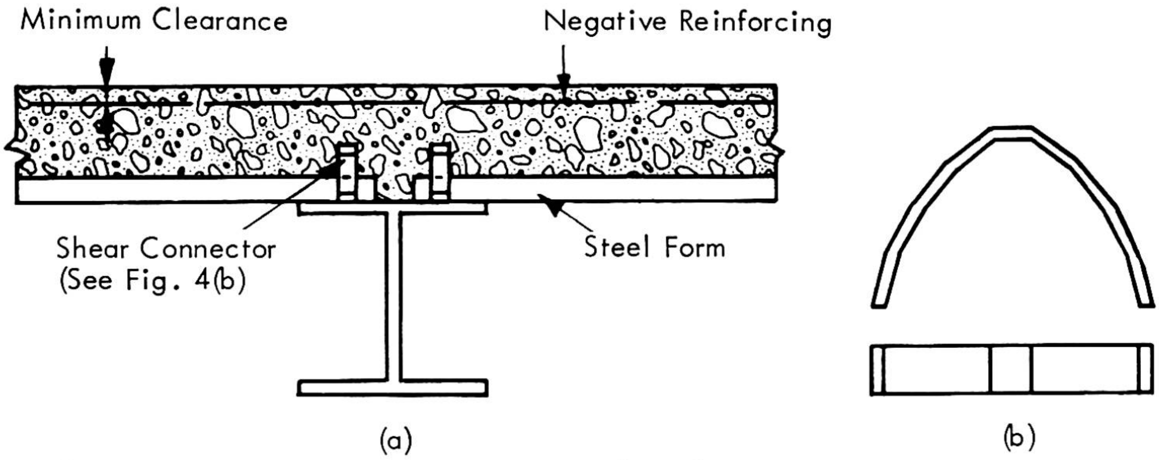

Fig. 4. Typical composite section beam detail.

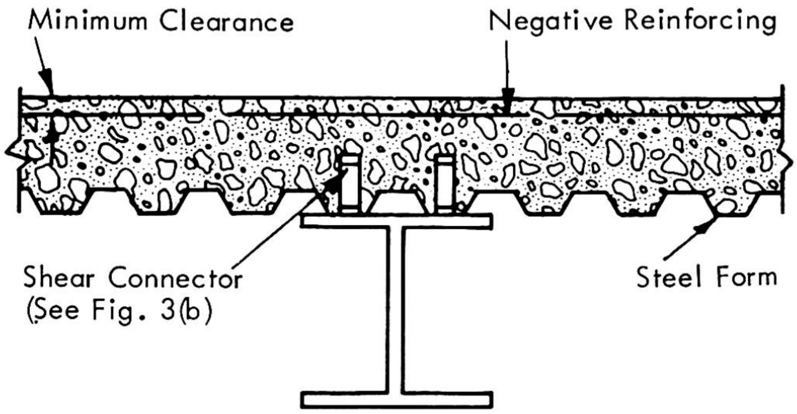

Fig. 5. Typical composite section girder detail.

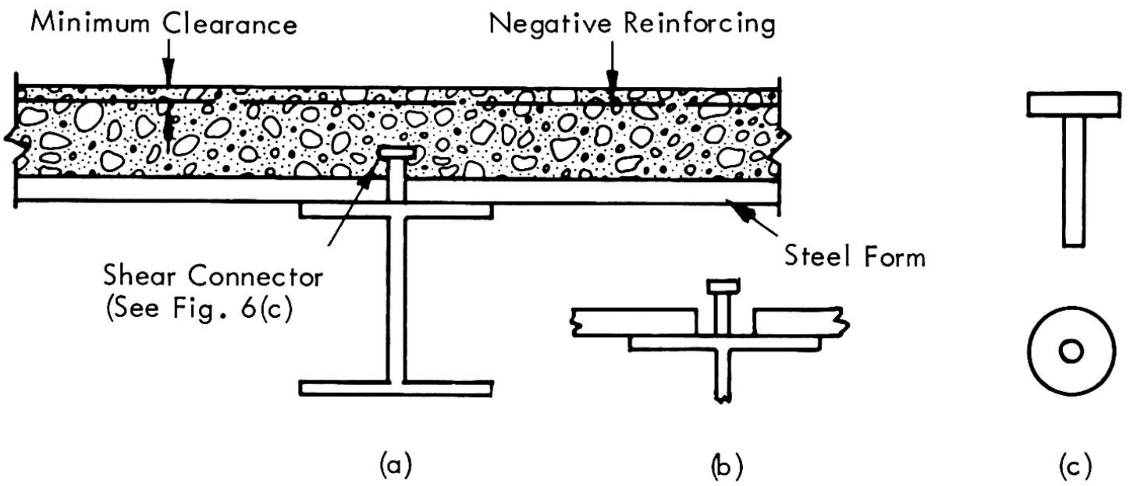

Fig. 6. Typical composite section beam detail.

ILLUSTRATIVE EXAMPLES

This section contains photographs of actual applications of light gage steel forms in buildings. Two types of forms are shown, Category I and Category II(b), as well as the installation of raceways or ducts. Each figure from 7 through 14, is shown with a descriptive title.

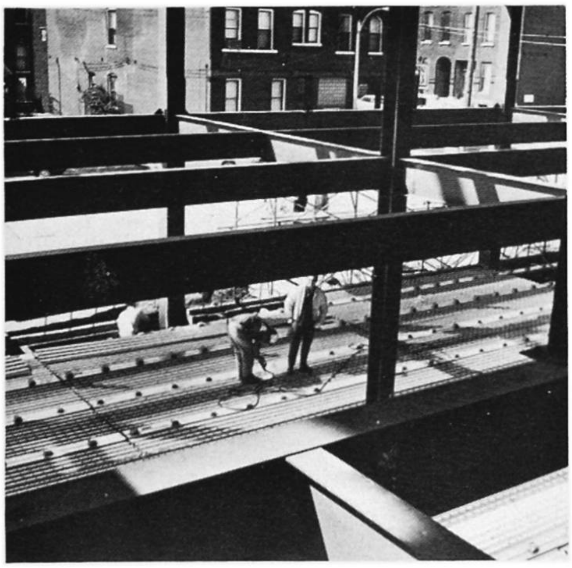

Fig. 7. Category I Form — Overview of installation showing raceways with uniformly spaced service fittings.

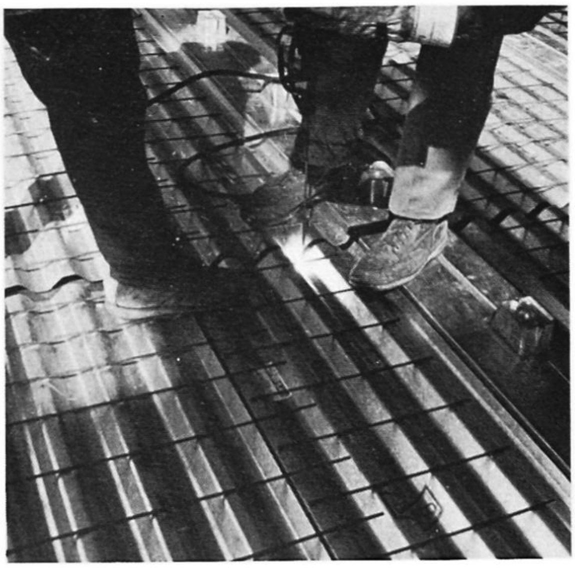

Fig. 8. Category I Form — Closeup showing form being fastened to structural frame.

Fig. 9. Category I Form — Overview showing all forms and raceways with service fittings in place.

Fig. 10. Category I Form — View showing placement of concrete. (Note negative reinforcing bars.)

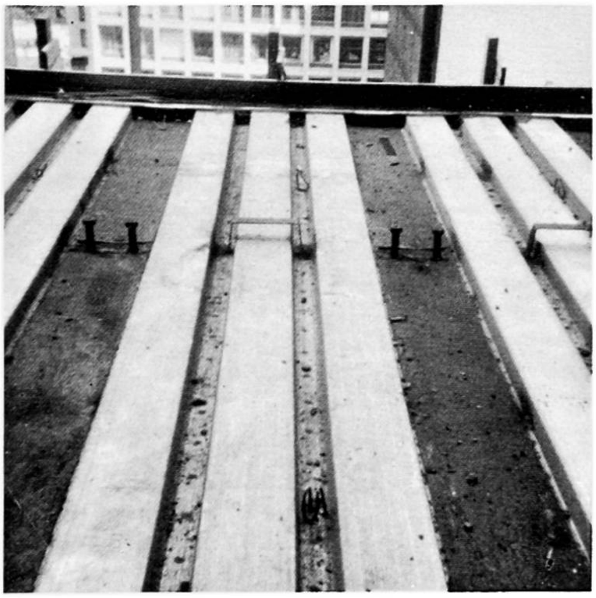

Fig. 11. Category II(b) Form – Special module designed to carry service wiring.

Fig. 12. Category II(b) Form – View showing raceways for wiring with uniformly spaced service fittings.

Fig. 13. Category II(b) Form – Overview showing composite beam and girder construction. (Note temperature steel.)

Fig. 14. Category II(b) Form – Application of spray-on fireproofing.

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SUMMARY

Floor systems with composite form-reinforced concrete slabs is a rapidly growing and developing method of construction. The utilization of composite action between slabs and supporting beams and girders is expected to increase correspondingly.

Further research and development is necessary to establish commonly accepted design criteria. This would ultimately improve conditions for further development of new concepts.

RÉSUMÉ

Les systèmes de planchers en profilés de béton armé renforcés d'acier en action combinée gagnent rapidement d'importance dans la construction. On peut donc s'attendre que parallèlement à ce développement, on s'intéresse de plus en plus à une action combinée entre le plancher et la structure maîtresse.

Il serait donc nécessaire de pousser les recherches et le développement dans ce domaine, afin d'établir des critères de projection communément admis. Cela permettrait à la suite d'améliorer les conditions pour le développement ultérieur et pour des conceptions nouvelles.

ZUSAMMENFASSUNG

Deckentragwerke (Leichtbleche) im Verbund mit Stahlbetonplatten sind eine rasch wachsende und entwicklungsfähige Konstruktionsmethode. Gleichzeitig wird die Anwendung des Verbundes zwischen Platten und Hauptträgern (Unterzüge) erwartet. Künftige Forschung und Entwicklung wird nötig sein, um allgemein anerkannte Entwurfskriterien zu erhalten. Dies würde die Bedingungen für weitere Entwicklungen verbessern.

IIb

Thin-Walled Steel Hyperbolic Paraboloid Structures

Structures en tôle paraboloïdes hyperboliques

Dünnwandige hyperbolische Paraboloid-Stahltragwerke

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USA

Attractive and economical hyperbolic paraboloid structures can be built of light gage steel elements (such as those shown in Fig. 1). An extensive investigation of the behavior of such structures is underway at Cornell University under the general direction of George Winter.

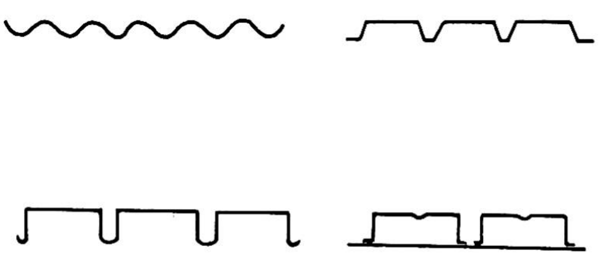

Fig. 1

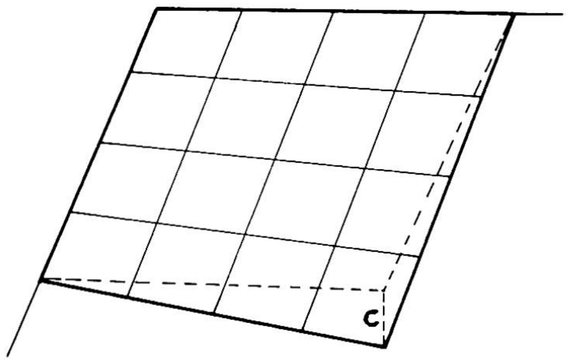

Fig. 2

Since a hypar is a warped surface generated by straight lines (Fig. 2), it can therefore be formed using light gage steel panels. These panels are laid side by side and warped individually. Welding or sheet metal screws are used to connect the panels to each other and the edges of the resulting shell decking to the edge members to form a hypar unit. Such units can be connected in a great variety of ways to produce interesting structures (Fig. 3). Often double layers are used with the deformations or corrugations of the two layers running perpendicular to each other. An example of a light gage hypar structure is shown in Fig. 4[1].

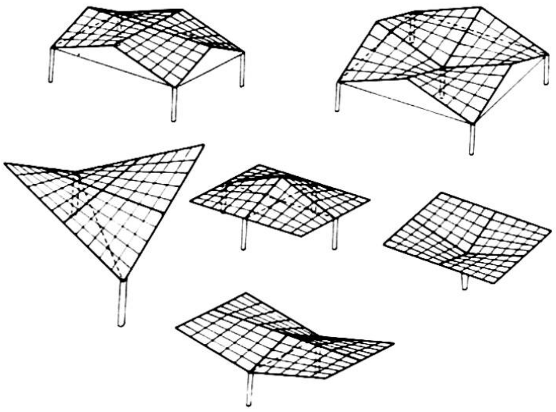

Fig. 3

Fig. 4

It is well known that, according to the membrane theory, hypars develop uniform shear stresses $S = pab/2c$ due to a uniform load p, where a and b are the side dimensions of the plan of a hypar unit and c is the rise of a corner with respect to an opposite edge as shown in Fig. 2. The equation describing the surface is $z = xyc/ab$. It was demonstrated in an extensive research program at Cornell University[2,3] (see also Report on Theme II/a) that light gage diaphragms can transmit uniform in-plane shear stresses. The ease of forming the surface and the satisfactory diaphragm action of the decking make the structure feasible and structurally efficient. With heavy decking and large edge member sizes, simple units of 50 ft by 50 ft (15 m by 15 m) or larger can be built. Thus, a 100 ft by 100 ft (30 m by 30 m) roof may be supported by a single central column. The dead load to live load ratio is exceptionally small for these structures.

Since the decking is connected at varying angles along the edge members, tubular edge members are convenient. However, a flange plate can be welded to other shapes and then warped (Fig. 5). For small structures edge members with an angle shape may be sufficient. The decking transmits uniform unit shear forces along the con-

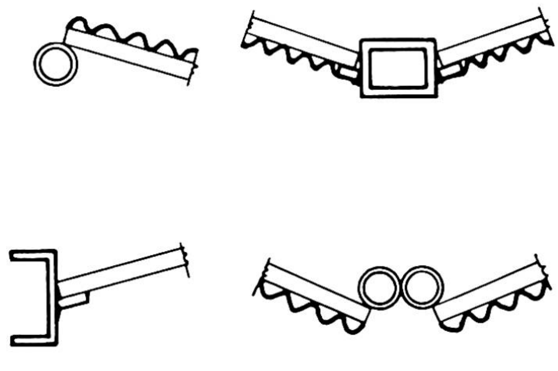

Fig. 5

nection, thus the edge member is loaded by an axial force varying linearly from zero at one end to a maximum tension or compression force at the other end.

Therefore, the preliminary design of light gage hypars consists of (1) the selection of the deck sizes to obtain the required rigidity, (2) the design of all the connections (either welds or screws) between the panels of the deck and between the deck and the edge members to transmit the membrane shears, (3) the design of the edge members. The forces in the edge members are easily calculated for the membrane stress condition. Care must be taken in applying the connections to insure that the welds or screws can take the shear forces.

The membrane stresses are usually small and the structure can carry remarkably heavy uniform loads. A small scale model of a hypar structure under loading is shown in Fig. 6.

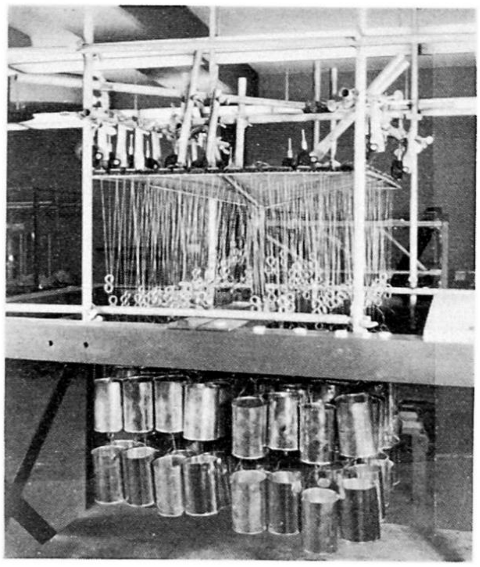

Fig. 6

The major problems in the design of light gage hypars involve deflections and buckling. The present investigation at Cornell University is concerned with these questions.

Light gage hypars may be rather flexible due to the small bending rigidity of the edge members and the possible low shear rigidity of the deck. Since both of these elements resist the deflections of the structure, it is not easy to determine the sizes required to limit deformations. For example, for an inverted umbrella shell (Fig. 7) the external corner deflection (A) is caused primarily by loads near this corner. This point shows practically no deflection for loads away from this corner. Usually the deflections of a free corner (such as point A) and the deflections of the center of the deck are of interest.

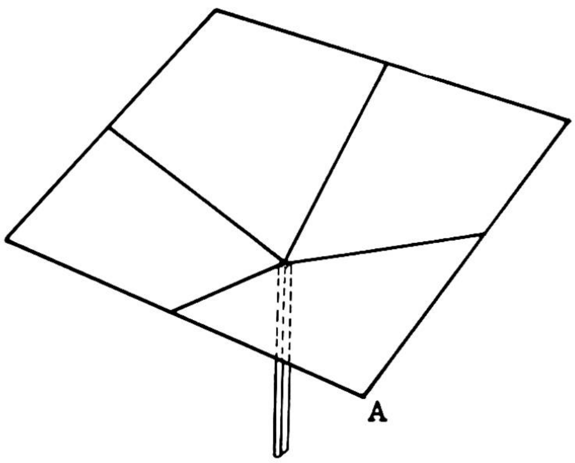

Fig. 7

It was found that for practical member sizes these deformations depend strongly on the effective shear rigidity of the deck (in addition to the proportions of the structure). The effective shear rigidity of a light gage steel deck is defined as the uniform edge force per unit length required to cause a unit angular distortion of the deck. Unless the rise (c) is relatively large (say $a/c < 2$), the shear rigidity obtained from flat diaphragm tests[3] may be used in the calculations.

The effective shear rigidity of light gage steel decking depends on several factors. If a single layer of sheet is used for the decking, the effective shear rigidity depends somewhat on the panel to panel connections and primarily on the spacing of the connectors (screws, welds) to the edge members (especially in the weak direction, i.e. transverse to the ribs or corrugations). It also varies with the plan dimensions. If two layers are used with the ribs of one layer running transverse to the ribs of the other layer, then the effective shear rigidity of the two layers also depends on the method of connection of the two layers to each other and to the edge members. If the edge members utilize warped flange plates for connection (Fig. 5) then each layer may be connected directly to the plate (one on each side of the plate) and the effective shear rigidity of the two layers in this case is approximately twice that for a single layer. However, if a single tubular edge member is used without flange plate, then only one layer of the decking can be attached directly to the tube and the second layer has to be attached to the first layer. In this case, the effective shear rigidity of the two layers is only about 50% larger than that for one layer because of the larger "eccentricity" of the shear being transferred to the edge members. Built-up box type edge members may be used so that each layer is connected directly to the edge members[4]. Box type edge members have the advantage of larger torsional rigidity.

The amount of interconnection of the two layers also strongly influences the shear rigidity of the deck. Tests were performed on 5 ft by 5 ft (1.5 m by 1.5 m) shells with continuously supported edge members. For each of the three rise to span ratios investigated (1/8, 1/5, 1/3) there was a 25% reduction in maximum deflections of the deck for the fully connected decks (sheet metal screws spacing equal to the pitch of the corrugated decking) as compared with the unconnected decks. It must be remembered, that it is usually necessary to interconnect the layers to some degree to prevent chatter due to wind.

If a single layer deck is used, large deformations may occur under localized loads since the deck cannot transmit the local loads by in-plane shear stresses and thus it has to distribute the load by plate bending along the ribs. Double layers are recommended if large local (concentrated) loads are expected. To illustrate: For each of the three rise to span ratios tested, the maximum deflection under a load covering 8 in by 12 in (20 cm by 30 cm) area at the center of the deck for two unconnected layers was only one third that for a single layer.

In the analysis of light gage steel hypars it is more important to separate the action of the edge members and the deck than it is in the analysis of reinforced concrete hypars. The eccentric forces on the edge members have to be considered. Two approaches have been followed in the investigation at Cornell University: the finite element method, and the numerical solution of the shallow shell equations together with the differential equations of flexure

$$R_1\partial^4F/\partial y^4 + R_2\partial^4F/\partial x^2\partial y^2 + R_3\partial^4F/\partial x^4$$
$$= Eh[(2c/ab)\partial^2w/\partial x\partial y + (\partial^2w/\partial x\partial y)^2 - (\partial^2w/\partial x^2)(\partial^2w/\partial y^2)]$$

$$D[R_4\partial^4w/\partial x^4 + R_5\partial^4w/\partial x^2\partial y^2 + R_6\partial^4w/\partial y^4] = - p$$
$$+ (\partial^2w/\partial x^2)(\partial^2F/\partial y^2) + (\partial^2w/\partial y^2)(\partial^2F/\partial x^2)$$
$$- 2(c/ab + \partial^2w/\partial x\partial y)(\partial^2F/\partial x\partial y)$$

and torsion for the edge members. Both of these approaches include the orthotropic properties of the shell. For the finite element procedure a twelve term polynomial and rectangular plate elements are used. The flexural rigidity of the edge members and the eccentricity of the edge shears are included.

For the calculation of the approximate deflection of the deck relative to simply supported edges the shallow shell equations or energy methods can be applied. However, the deflection of the edge members under the eccentric tangential shear forces is complicated and thus the above numerical-computer procedures are used. As a result of these analyses, the required deck sizes and connection spacings can be determined to limit deflections and to achieve the necessary strength.

Buckling of hypar structures may occur in one of two ways: either the deck buckles and the edge members remain essentially straight; or the compression edge members may buckle which actually means the collapse of the structure. Strictly speaking the buckling of the edge members is an inelastic beam-column deformation or stress phenomenon rather than an eigenvalue problem. However, the failure is abrupt. For example, an inverted umbrella roof (Fig. 7) may suddenly turn inside out.

The hypar surface is made up of two sets of orthogonal parabolas, one set behave as arches, the other as cables. If the load reaches the critical value, the shell buckles in a number of waves along the compression arches (Fig. 8). The buckling load for uniform loading and simply supported isotropic shells (rigid edge members) was calculated by Reissner[5]. His theory was generalized for orthotropic shells to be applicable for light gage steel hypars. The resulting equation has the following form

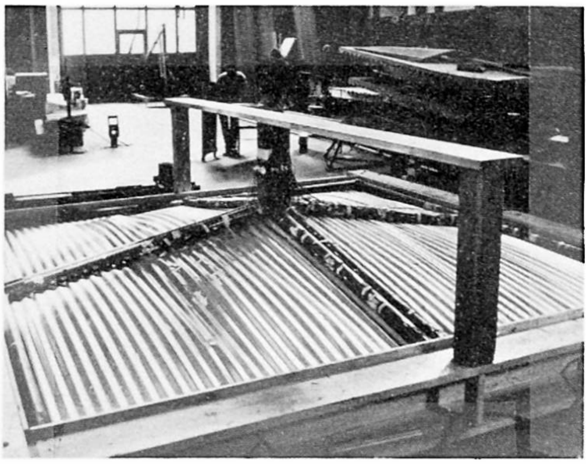

Fig. 8

$$p_{cr} = 4(c/ab)^2 \sqrt{EhD}\sqrt{(R_4+R_5+R_6)/(R_1+R_2+R_3)}$$

where h is the thickness of the shell, E is the modulus of elasticity, D is the plate flexural rigidity, and the constants R depend on the shear and flexural rigidities of the orthotropic deck. For isotropic shells $R_1+R_2+R_3 = R_4+R_5+R_6 = 4$. It was found that the boundary conditions and the edge member flexural rigidity about either axis are not very significant. According to the work by Leet[6] on plastic models, the axial stiffness of the edge member does have an appreciable influence on the initial membrane buckling load. The effective shear rigidity of the deck is a very important factor and must be known with reasonable accuracy.

Considerable post-buckling strength exists provided that the deck is well fastened to the edge members which must have sufficient bending rigidity in the plane tangent to the shell at the edge; in such a case an appreciable tension field can develop in the shell along the ribs.

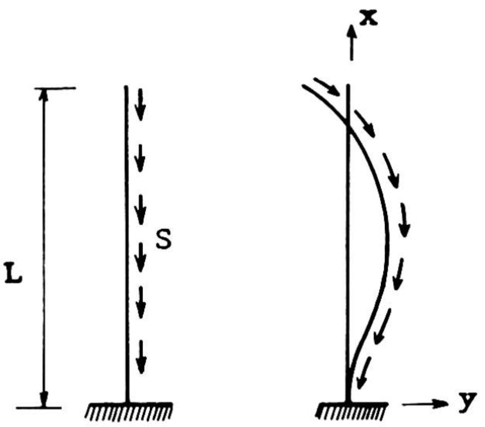

Fig. 9

The "buckling" of the compression edge members is caused by the eccentric tangential shear forces. The stability of a column under the so-called "follower forces" (Fig. 9) is a non-conservative problem and dynamic analysis is necessary. The differential equation is $EI.\partial^4 y/\partial x^4 + S(L-x)\partial^2 y/\partial x^2 + m\partial^2 y/\partial t^2 = 0$. The results of such calculations for various limiting boundary conditions were found to bracket the experimental collapse loads for the deck and edge member sizes tested.

The finite element method or the numerical solution of the orthotropic equations give the load-deflection curves and also the smaller of the deck buckling or collapse loads. Therefore, design curves can be prepared based on deflection and stress limitations and on adequate safety factors against deck buckling or collapse. Extensive use of light gage steel hyperbolic paraboloid structures is expected in the future for such applications as service stations, small office buildings, vacation homes, factories, schools and canopies.

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Zur Optimierung der Höhe von Balkenbrücken mit Hohlkastenquerschnitt

The Optimum Height of Steel Closed Continuous Girder Bridges

La hauteur optimum des ponts continus à longerons de section ferrée en acier

S. A. ILJASEWITSCH
Prof. UdSSR

Wir betrachten den einfachsten Kastenquerschnitt mit:

- b- Breite;
- H - Höhe;
- δ - Horizontalblechdicke (Gurtbleche);
- δ_c - Vertikalblechdicke;
- n - Anzahl der Vertikalbleche.

Bestimmen wir die Optimalgröße für H_{optR}, die bei voller Ausnutzung der rechnerischen Stahlfestigkeit - " R " in den Querschnitt - Gurtblechen dem Mindestverbrauch an Baustahl entspricht.

Die Formel für Bestimmung des Konstruktionsgewichts pro lfd.M. der Brücke lautet:

$$g = \frac{2M\beta\psi_1}{HR}\gamma + n\delta_c H\psi_2\gamma + g_H \tag{1}$$

wobei:

M -Gesamtbiegemoment;

$\beta = 1 - \frac{J_c}{J}$ - J_c und J - die Trägheitsmomente der Wände und Kastenquerschnitte sind;

ψ_1, ψ_2 - Beiwerte für Berücksichtigung der Gewichtszunahme von Gurtblechen und Kastenwände bei der Ausführung $(\psi_1 \sim 1{,}4;\ \psi_2 \sim 1{,}25)$;

γ - Raumgewicht von Stahl;

g_H - Gewicht pro lfd. M. des Fahrbahnbelags.

Nehmen wir an, daß die von H und der Anzahl der waagrechten Versteifungsrippen i abhängige Wanddicke δ_c die örtliche Wand-Stabilität gewährleistet.

Hieraus:

$$\delta_c = 1{,}3\sqrt{\frac{R(1-\mu^2)}{Ek}} \cdot \alpha H = \xi_R \alpha H,$$

Wobei:

$$\xi_R = 1{,}3\sqrt{\frac{R(1-\mu^2)}{Ek}}; \quad \alpha = \frac{1}{i+1}; \quad \beta = 1 - \frac{n\xi_R \alpha H^4}{12}; \quad k = 4{,}0\sqrt{\frac{2{,}1}{R}}$$

Nach entsprechenden Substitutionen und Transformationen läßt sich (1) wie folgt schreiben :

$$g = \frac{2M\psi_1}{HR}\cdot\gamma + n\xi_R \alpha H^2\left(\psi_2 - \frac{\psi_1}{3}\right)\gamma + g_H \tag{2}$$

Die der Gleichung (2) entsprechende Kurve hat ein bestimmtes Minimum. Bei anderen den Wert R über- bzw. unterschreitenden Spannungs-größen, die gleich σ_1, σ_2, σ_3 ...sind, lassen sich ähnliche Kurven, die in anderen parallel verlaufenden Ebenen liegen, auftragen. Rwatschow Ju.A. und Salamachin P.M. haben dabei gezeigt, daß die dem Minimum diesen Kurven entsprechenden Stellen auf einer Gerade liegen. Diese Minima stellen die Optimalwerte der Höhen für verschiedenen Varianten dieser Querschnittsform fest . Jeder Optimalhöhe ein bestimmter Wert der relativen Durchbiegung entspricht, den man begingt als optimal bezeichen kann. Die im Bild 1 dargestellten Kurven wurden als Isotensen genannt.

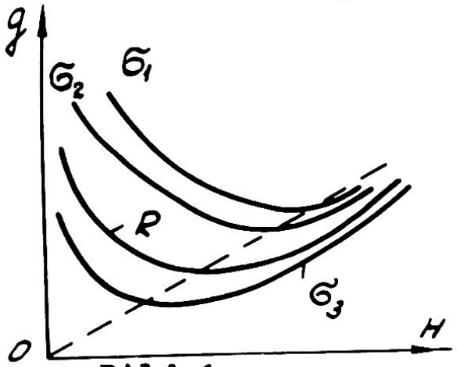

Bild 1

Mit der Gleichung (2) bestimmt man die Optimalwerte der Höhe $H_{opt,\sigma}$ unter voller Ausnutzung der Normalspannung:

$$H_{opt,\sigma} = \sqrt[3]{\frac{M}{1{,}3Rn\alpha\left(\frac{\psi_2}{\psi_1} - \frac{1}{3}\right)}}\sqrt{\frac{Ek}{R(1-\mu^2)}} \tag{3}$$

Da $f = \frac{2Rl^2}{AEH}$

und " A " der vom Balkensystem und der Beanspruchungsart abhängige Faktor ist, so ergibt sich daraus:

$$\left(\frac{f}{l}\right)_{opt,\sigma} H_{opt,\sigma} = \frac{2Rl}{AE} = \eta_\sigma \tag{4}$$

Ausgehend von der Blechberechnung nach der zulässigen Durchbiegung $\left[\frac{f}{l}\right]$ bei der Gurtspannungen, die gleich sind, finden wir

$$\sigma = \frac{AE}{2l}\left[\frac{f}{l}\right]H = \eta_f H; \qquad \eta_f = \frac{AE}{2l}\left[\frac{f}{l}\right]$$

In diesem Falle ist in (1) anstatt $R, \xi_R, \delta_c, \beta$ –

$$\sigma = \eta_f H; \quad \xi_f = 1{,}3\sqrt{\frac{\eta_f H(1-\mu^2)}{Ek}}; \quad \delta_c = \sqrt{\frac{\eta_f H^3(1-\mu^2)}{Ek}} = \varepsilon\alpha H^{3/2}; \quad \beta = 1 - \frac{n\varepsilon\alpha H^{9/2}}{12J}$$

einzusetzen.

Nach entsprechenden Substitutionen und der einfachsten Transformationen Gleichung (1) läßt sich die in folgender Form schreiben:

$$g = \frac{2M\psi_1}{\eta_f H^2}\gamma + n\varepsilon\alpha\gamma\left(\psi_2 - \frac{\psi_1}{3}\right)H^{5/2} + g_H \qquad (5)$$

Die betreffende Kurve wurde als Isoflexe bezeichnet. Sie hat ein dem minimalen Konstruktionsgewicht entsprechenden Minimum, ohne sich von Isotense durch den Gesamtansicht zu unterscheiden. Für verschiedene Werte der Relativdurchbiegungen gibt es ähnliche Kurven mit eigenen Minima. Wie es bei Isotensen der Fall ist, so liegen auch die dem Isoflex-Minumum entsprechende Stellen auf einer Gerade. Das Isoflex-Minumum bestimmt den Optimalwert der Höhe $H_{opt,f}$ bei der Berechnung der Konstruktion ihrer Durchbiegung nach:

$$H_{opt,f} = \sqrt[9]{\left[\frac{1{,}6M}{\eta_f n\varepsilon\alpha\left(\frac{\psi_2}{\psi_1} - \frac{1}{3}\right)}\right]^2} \qquad (6)$$

Von praktischem Interesse sind die Werte von $H_{opt,f}$, die für den Fall der zulässigen relativen Grenzdurchbiegung $\left[\frac{f}{l}\right]$ bestimmt werden. In diesem Falle eine bestimmte Gurtspannung, die bedingt als optimal bezeichnet werden kann und beträgt:

$$\sigma_{opt,f} = \eta_f H_{opt,f} = \frac{AE}{2l}\left[\frac{f}{l}\right]H_{opt,f} \qquad (7)$$

Bei einigen Querschnitten aber hat die Querkraft eine grundlegende Bedeutung.

In diesen Fällen bei geringer relativer Wanddicke sind die Wände nicht imstande, die beträchtlichen Normalbiegespannungen aufzunehmen. Dann kann man von der Bedingung ausgehen, daß das Moment auf die Gurte und die Querkraft beziehungsweise auf die Wände übertragen wird.

Daraus ergibt sich:

$$\delta_c = \sqrt[3]{\frac{Q(1-\mu^2)}{0{,}59\,\nu n k_\tau E}} \cdot H^{1/3}$$

wobei

"K_τ " ist dabei der vom Verhältnis des Abstandes zwischen vertikalen Versteifungsrippen zu Wandhöhe sowie von der Befestigungsart der Wandränder abhängige Faktor (er ist tabellarisch dargestellt;

$\nu = \frac{J}{SH}$ und S- statisches Moment des halben Querschnitt $(\nu \sim 0{,}9)$

Für lfd M. der Brückenkonstruktion, bei der die rechnerischen Normal- und Schubgrenzspannungen maximal ausgenutzt werden, läßt sich somit

$$g = \frac{2M\phi_1}{HR}\gamma + n\phi_2\gamma\sqrt[3]{\frac{Q(1-\mu^2)}{0{,}59\,\nu k_\tau n E}} \cdot H^{4/3} + g_H \tag{8}$$

schreiben. Minimisierung dieser Funktion ergibt für Optimalhöhe eines Kastenquerschnitts, der unter Ausnutzung von Grenzfestigkeit von Stahl wie in den Gurten (Normalspannung) als auch in den Wänden (Schubspannung τ) berechnet wird, die Formel:

$$H_{opt,\sigma,\tau} = \left(\frac{3M\phi_1}{2Rn\phi_2}\right)^{3/7} \cdot \left[\frac{0{,}59\,\nu n k_\tau E}{Q(1-\mu^2)}\right]^{1/7} \tag{9}$$

Bei der Projektierung der Konstruktionen ausgehend von der Bedingung der vollen Ausnutzung von zulässiger relativer Grenzdurchbiegung $\left[\frac{f}{l}\right]$ sowie von rechnerischem Schubwiderstand der Wand, nimmt die entsprechende Optimalwert H_{opt}, τ, f :

$$H_{opt,\tau,f} = \left(\frac{6M\phi_1 l}{AEn\phi_2}\left[\frac{l}{f}\right]\right)^{3/10} \cdot \left[\frac{0{,}59\,\nu k_\tau n E}{Q(1-\mu^2)}\right]^{1/10} \tag{10}$$

Die Isotense, die die Varianten der Konstruktion mit voller Ausnutzung der rechnerischen Grenzfestigkeit R des Stahls in den Gurten kennzeichnet, als auch die den Varianten mit zulässiger Relativgrenzdurchbiegung $\left[\frac{f}{l}\right]$ entsprechende Isoflexe, wie zuerst Sacharow W.W. und dann Rwatschow Ju.A. und Salamachin P.M. gezeigt haben, können sich (in verschiedenen Ebenen liegend) gegenseitig in drei unterschiedlichen Positionen befinden, die von Interesse sind. Die erste von ihnen kennzeichnet sich durch das Vorhandensein von wirklichen Schnittpunkt der Isotense mit Isoflexe, links von den Stellen, die den Minimalwerten dieser Kurven entsprechen.

Bei der zweiten Position liegt der Schnittpunkt der Isotense mit Isoflexe zwischen den Minimalwerten jeder der beiden Kurven. Die dritte Position wird durch die Existenz des Schnittpunktes rechts von Mindestwerten der Isotense und Isoflexe charakterisiert. Diese drei verschiedenen gegenseitigen Positionen der betrachteten Kurves legen drei mögliche Bereiche der Projektierung fest. Nehmen wir an, daß Projektierung ausgehend vom Konstruktionsfestigkeitnachweis durchgeführt wird.

Tragen wir auf die gegebene Isotense (Bild 2) zwei Isoflexen auf, die eine von denen, die der Relativdurchbiegung $\left(\frac{f}{l}\right)_1$ entspricht, durch das Isotense-Minimum verläuft, und die andere, die Varianten mit Relativdurchbiegung $\left(\frac{f}{l}\right)_2$ einschließende Isoflexe sich mit Isotense in der Stelle des Isoflexe-Minimums schneidet. Durch diese zwei Isoflexen werden drei Projektierungsbereiche festgelegt. Der erste Bereich befindet sich links von der ersten Isoflexe und wird durch das Vorhanden sein (bei den Varianten dieses Bereichs) der Relativdurchbiegungen, die die relative Durchbiegung der Variante mit Mindestgewicht überschreiten, gekennzeichnet.

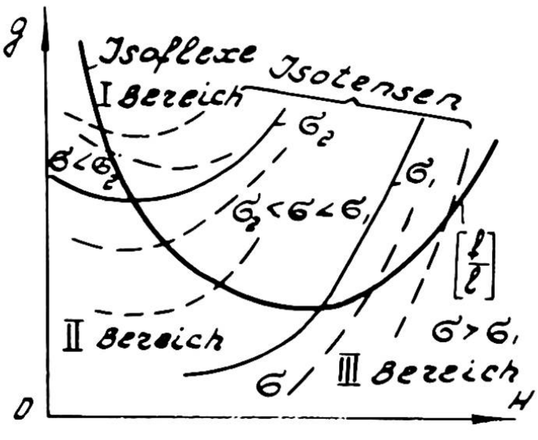

Bild 2

Der zweite Projektierungsbereich enthält die Varianten der gegebenen Isotense, die durch die Werte von Relativdurchbiegungen, die $\left(\frac{f}{l}\right)_1$ unter - bzw. $\left(\frac{f}{l}\right)_2$ überschreiten, charakterisiert werden. Beim dritten Bereich kennzeichnen sich die Varianten durch die Relativdurchbiegungen, die den Wert $\left(\frac{f}{l}\right)_2$ unterschreiten.

Bei der Projektierung einer Konstruktion von der Bedingung ihrer Durchbiegung ausgehend, können ebenfalls ähnliche drei Bereiche der Projektierung bezeichnet werden. Die Grenzen dieser Bereiche werden durch die Schnittpunkte der gegebenen Isoflexe mit zwei Isotensen bestimmt. Die eine von ihnen, die die Varianten mit der Spannung σ_1, enthält, schneidet die Isoflexe in der Stelle des Isoflex-Minimums durch; die andere die Varianten mit der Spannung σ_2 einschließende Isotense schneidet die Isoflexe mit Isotense-Minimum durch. (Bild 3). In diesem Fall liegt der erste Projektierungsbereich oberhalb der der Spannung σ_2 entsprechenden Isotense; dieser Bereich charakterisiert die Varianten, die sich durch die volle Ausnutzung einer und derselben zulässigen Relativ-Grenzdurchbiegung $\left(\frac{f}{l}\right)$ sowie durch die σ_2 unterschreitenden Spannungen σ auszeichnen. Die Varianten dieses Bereichs mit der Spannungen, die σ_1 unter- bzw. σ_2 überschreiten, charakterisieren sich durch das Vorhandensein der gleichwertigen zulässigen relativen Grenzdurchbiegung $\left[\frac{f}{l}\right]$ ebenfalls. Der dritte Bereich befindet sich rechts von der Isotense mit der Spannung σ_1. Die betreffenden durch die gegebene Isoflexe festlegenden Varianten haben dabei die Spannungen σ, die σ_1 überschreiten.

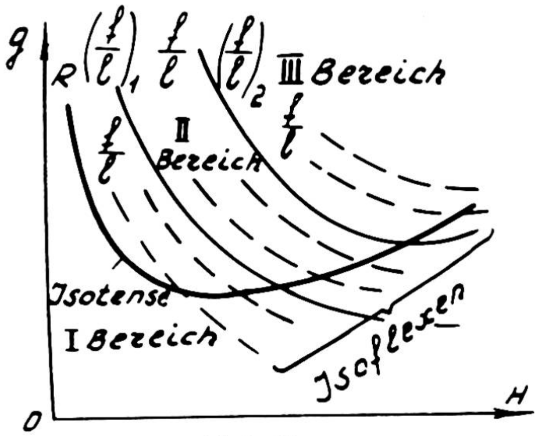

Bild 3

Die oben erwähnten Werte sind bei dem Festigkeitsnachweis nicht nur in Beziehung auf die für einen Werkstoff (bei einem bestimmten Wert von " R ") ermittelten Isotensen, sondern auch in Hinsicht auf die anderen, den Werkstoffen mit unterschiedlichen Rechenwerten der Grenzfestigkeit entsprechenden Isotensen zu finden. Bei dem Steifigkeitsnachweis ist es notwendig, über analoge Angaben nicht nur für die sich durch einen Wert der zulässigen relativen Grenzdurchbiegung $\left[\frac{f}{l}\right]$ charakterisierende

Isoflexe, sondern auch für verschiedene Isoflexen, die bei anderen Werten von $\frac{f}{l}$ bestimmt werden, zu verfügen. Das Vorhandensein der erwähnten Angaben erlaubt die gerechtfestigte zweckmäßigste Variante auszuwählen.

Aber schon die Ermittlung von nur einer Isotense bzw. Isoflexe erfordert einen großen Rechenaufwand. Bei der Lösung der Gesamt-Aufgabe aber hat man, wie es die obenangeführten Überlegungen beweisen, mit einer größeren Anzahl von Isotensen bzw. Isoflexen zu tun. Bei diesen Verhältnissen ist es erforderlich ein so großes Ausmaß an Rechenarbeit zu bewältigen, so daß diese umfangreiche Arbeit bei der Handberechnung nicht als praktisch annehmbar erachtet werden kann.

Etwas günstiger liegen die Verhältnisse bei der Ausführung des obengeführten Rechenaufwands mit Hilfe von elektronischen Rechenanlagen. Mit einem speziell für die betreffende elektronische Digitalrechenanlage ausgearbeiteten Programm wird die untersuchte Aufgabe mit grösstmöglicher Vollständigkeit und Exaktheit gelöst.

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ZUSAMMENFASSUNG

Anhand der Untersuchung von zwei Typen der Gewichtskurven, die als Isotense und Isoflexe bezeichnet werden, wird eine Analyse der drei möglichen Projektierungsbereiche (ausgehend von der Festigkeit, der Festigkeit und Steifigkeit, sowie von der Steifigkeit) durchgeführt.

Diese Projektierungsbereiche werden in Abhängigkeit von der Art der Gegenseitigen Lage von Isotensen und Isoflexen festgelegt. Durch diese Analyse wird die Möglichkeit der exakten Ermittlung der gewichtsmäßig optimalen Konstruktionen gewährleistet.

SUMMARY

An analysis based on the study of two types of weight curves (isotenses and isoflexes) is presented for three possible areas of designing (strength analysis, strength and stiffness analysis and stiffness analysis). These areas are determined depending on the disposition of isotenses and isoflexes respecting each other.

The analysis provides the possibility of structural minimum weight optimization.

RÉSUMÉ

L'analyse des trois zones possibles de projection (suivant la résistance, résistance et rigidité, rigidité) est basée sur l'étude de deux types de courbes de poids (isotenze et isoflexe). On détermine ces zones en fonction du caractère de disposition mutuelle des isotenzes et isoflexes.

On peut trouver à l'aide de cette analyse des constructions dont le poids serait optimum.

Remarques de l'auteur du rapport introductif
Bemerkungen des Verfassers des Einführungsberichtes
Comments by the author of the introductory report

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The discussions have presented additional data on the many varied applications of light-gage cold-formed steel for floors and buildings.

From a study of all the papers presented, I believe that we shall see more developments in prefabricated mass produced components in the coming years. Floor systems and entire building frameworks are being investigated for fabrication feasibility and economical costs.

Currently, roof joists using chords of cold-formed steel and beams fabricated with a fluted web welded to the cold-formed chords are being mass produced and used as roof beams competitively with other types of sections.

The most recent innovation for the application of light-gage cold-formed steel is in the construction of unitized boxes. These boxes are planned to be prefabricated of room size, fully furnished and stacked in some manner to be a hotel, hospital, apartment or office building. At present, many different studies are in the development stage and no doubt many economical methods will evolve.

The development of stainless steel for structural applications is currently being researched at Cornell University under the direction of Dr. George Winter and the results of the early studies have already been incorporated in a design specification for austenitic steels. This 1968 specification "Design of Light Gage Cold-Formed Stainless Steel Structural Members" is published by the American Iron and Steel Institute and is comparable to the specification for carbon and low-alloy steels.

I feel confident that the future will see more structural components fabricated from cold-formed carbon, low-alloy and stainless steels.

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