Composite Design with Lightweight Aggregates on Building Projects

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COMPOSITE DESIGN using concrete slabs, steel beams and girders with various types of shear connectors has been used with some limitations for the past thirty or thirty-five years. Most of the applications of composite construction have been for highway bridge construction and only since the late 1940's and early 1950's has its application been accelerated in the building field.

Increasing use of ultimate strength design methods, development of various types of shear connectors, and extensive research in composite design in the past twelve years has advanced this type of construction and has allowed the structural designer to justify savings over conventional elastic and non-composite techniques.

This paper centers itself around building projects using composite design with a type of concrete that is unusual since it is lightweight with densities ranging from 90 to 110 pcf instead of the normal density of 150 pcf. This type of concrete has structural characteristics and physical properties similar to normal weight concrete.

Revisions to the American Institute of Steel Construction Specification in 1961, the American Concrete Institute Code in 1963, and earlier editions of the American Association of State Highway Officials Specifications recognized composite design with normal weight concrete, but these specifications have not yet introduced a design standard for the use of lightweight concrete in composite design. This type of design, however, has been used in the past ten years on several bridges and building projects with the individual designing engineer's modified criteria based on experience and accepted theories of composite design for normal weight concrete as defined in the above codes.

Detailed discussion of two specific building projects designed by Giffels and Rossetti, in which composite design with steel and lightweight concrete was used, is presented to illustrate the design and construction technique employed for this type of construction even in the absence of design standards and codes.

These projects—the Cobo Hall in the Detroit Civic Center (see Fig. 1) initiated in design in 1954, and the Detroit U. S. Post Office (see Fig. 2) designed in 1958—were actually developed at a time when both lightweight concrete and composite design in building construction were in their infancy.

This paper will also attempt to point out some of the shortcomings of the latest revisions to the AISC, ACI and AASHO specifications in that they do not include provisions for composite design with lightweight concrete.

THE DETROIT CIVIC CENTER

The Detroit Civic Center, which includes Cobo Hall and the Arena, a project with a budget in the order of 50 million dollars, was developed by the extensive use of long spans in structural steel. Many structural framing systems using large quantities of lightweight concrete as well as many structural systems of normal weight concrete were employed.

Cobo Hall, the largest and most important building of the Civic Center, covers over twelve acres with 1.6 million sq ft of floor, parking deck, meeting rooms and dining space.

The main exhibit hall (see Fig. 3) covers a total exhibit floor area of approximately 300,000 sq ft. It has only ten columns, in two rows of five columns each, spaced 240 ft apart transversely and 120 ft longitudinally. This exhibit area can be subdivided into three halls, **A, B,** and **C,** of 100,000 sq ft each, by vertical lift partitions retracted from the roof structure (see Fig. 4). Cobo Hall also has a secondary exhibit area of 100,000 sq ft, below the main floor, designated as Hall **D,** which supplements the main exhibit area at the upper level.

The six-lane John C. Lodge Expressway passes diagonally under the main exhibit floor of Cobo Hall to join the Chrysler Freeway.

The roof over the main exhibit area of Cobo Hall serves as a parking deck for 1300 cars. Lightweight con-

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Fig. 1. Cobo Hall view looking east. Note roof parking deck and John C. Lodge Expressway passing under hall.

Fig. 2. Detroit U. S. Post Office, view looking west; Fort Street elevation.

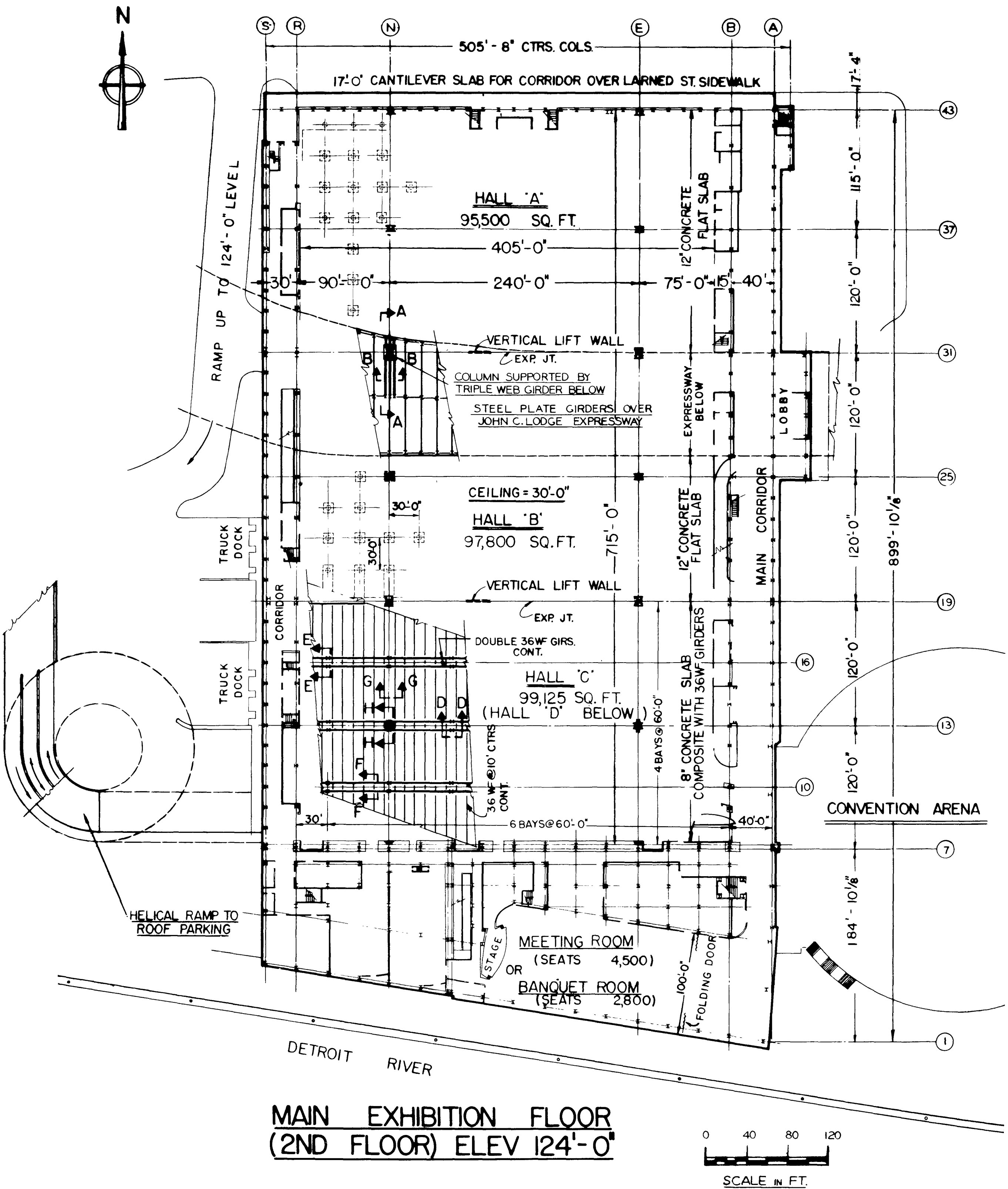

Fig. 3. Cobo Hall, main exhibit floor framing plan.

Fig. 4. Cobo Hall, Halls A, B, and C, view looking north. Note vertical lift doors partially open dividing the hall in three sections.

crete using permanent metal forms supported on beams and 240-ft clear span trusses were used for the roof framing system.

Design criteria required all exhibition hall floors at the main level to be designed to carry live loads of 1000 psf on concrete slab spans, 300 psf full area to beams, 200 psf full area to girders, or concentrated loads such as heavy machinery of 80 tons on a supported area 8 ft x 10 ft carried on at least two floor beams.

Framing for Hall C—(See Fig. 5). Hall **D,** with its 60 ft x 60 ft bays and heavy floor loads from Hall **C** above, required special floor framing to maintain a ceiling height of 18 ft-6 in. and a total structural depth limited to 44 in.

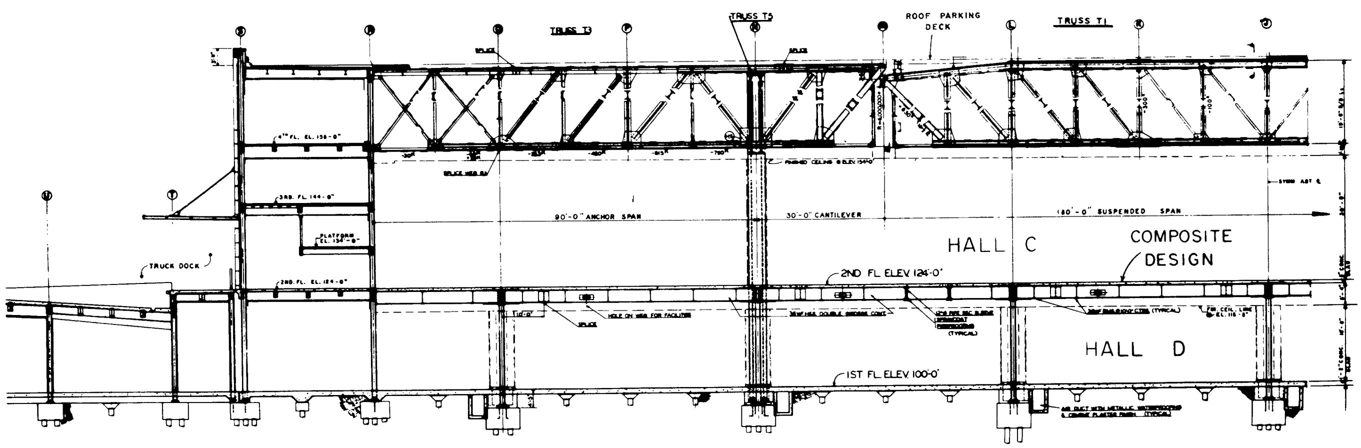

Fig. 5. Cobo Hall, typical cross section.

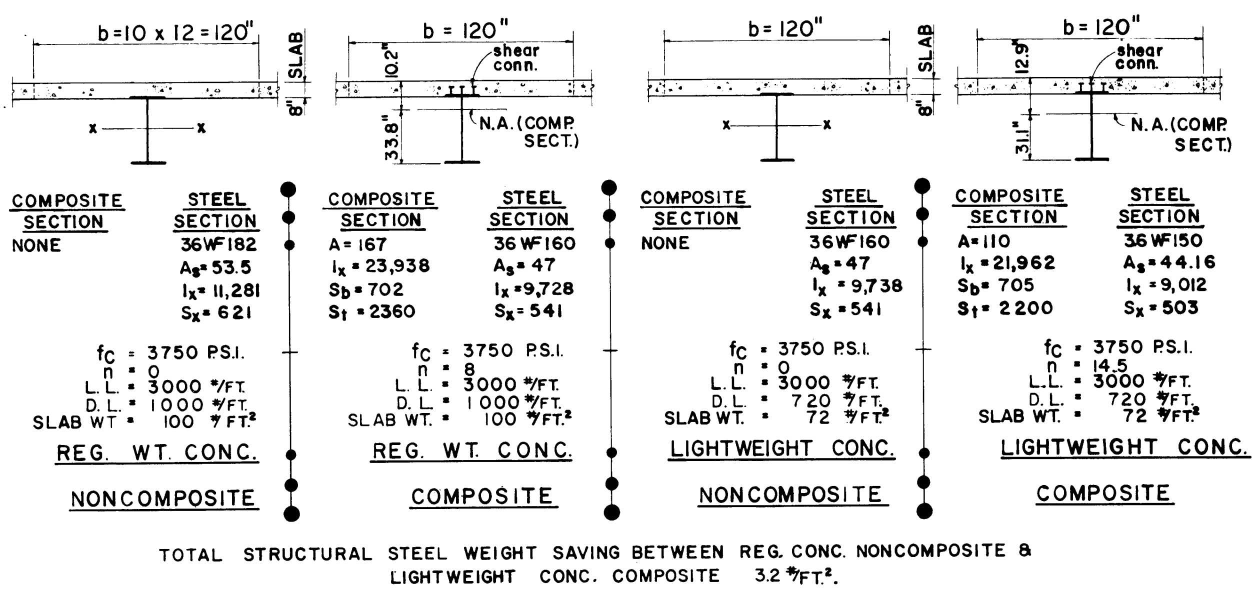

Fig. 6. Framing study for composite design. Cobo Hall exhibit floor—Hall C.

Many structural systems were studied for the exhibit floor system. The final decision to use steel framing in composite design with lightweight concrete was predicated on the ability of the structural system to carry the heavy floor loads economically on the large spans within the limited structural depth. The use of composite design with lightweight concrete resulted in a saving of approximately 3.2 psf of steel for Hall **C** (see Fig. 6) and also a reduced cost in the pile foundations.

To maintain acceptable limits of deflection with comparatively shallow construction for the 60-ft spans and the heavy live loads, the steel framing was designed for continuity in both directions and for composite action with the 8-in. thick lightweight concrete slab in the positive moment zones of the floor beams and girders. Composite action was not developed in the negative moment areas of the framing system.

The floor beams, spaced at 10-ft centers and spanning 60 ft were limited to a depth of 36 in. and were spliced at the girders with top and bottom flange plates for continuity. The double floor girders spaced at 60-ft centers and spanning 60 ft consisted of double 36WF beams of high strength steel with a minimum yield of 45,000 psi. These girders were also made continuous over supporting floor columns as shown in Fig. 7.

The stud type shear connectors proved to be the most economical type of connector studied. The ⅞-in. dia. shear connectors were welded to the top flanges of the floor beams and girders in the field after the erection of steel and installation of formwork.

Framing Over Expressway—The two-span girders over the six-lane expressway vary from 50 ft to 70 ft-6 in. in each span. The variation in span length is brought about by the expressway passing under the building in a wide sweeping curve as shown in Fig. 3. The framing for Cobo Hall in this area consists typically of 48-in. riveted steel plate girders placed on 15-ft centers designed in composite with the 9-in. lightweight concrete slab and continuous (with cover plates) over the center row of columns in the median strip separating the two directions of expressway traffic.

Fig. 7. Steel framing, Hall C, view looking north. Note 60 ft span beams framing into double girders.

The curved route of the expressway also made it necessary to transfer the load of one of the main roof columns to a triple web girder of high strength steel spanning 60 ft over the expressway. This column has a load of over 4 million pounds.

Composite Design Technique with Lightweight Concrete—In the absence of design standards for composite design with lightweight concrete at the time of design (1955–1957), the general design technique was developed around the AASHO standards, available publications on composite design by authorities in the field, and from test data by shear connector manufacturers. However, very little design or test data on composite design with lightweight concrete was available even in these publications.

In general, the composite section for the floor beams of Hall **C** and the plate girders over the expressway (Hall **B**) were designed on familiar elastic concepts, assuming that the concrete slab and steel beam would act as a unit up to ultimate load. The modular ratio n was computed to be 14.5 for the lightweight concrete. This ratio, using a steel modulus of elasticity E_s of 29 x 10^6 and a lightweight concrete modulus E_c of 2 x 10^6, was developed as follows:

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{2 \times 10^6} = 14.5$$

The modulus of elasticity of lightweight concrete as recommended by many authorities at the time of design was approximately 55 percent of normal weight concrete. The higher n-values for lightweight concrete mean smaller transformed areas than normal weight concrete and therefore smaller moments of inertia resulting in greater stresses and deflections. This effect is somewhat offset by the reduction in the dead weight afforded by lightweight concrete.

The horizontal shear between the concrete slab and the steel was computed by acceptable standard formulas based on the shear diagram between the points of contraflexure of the continuous beams and girders. Shear connectors were spaced according to the intensity of horizontal shear with the shear connectors closely spaced near the ends of the spans and with maximum spacing at the center of span.

Individual shear connector capacity was determined from semi-empirical formulas as developed by AASHO and based upon the concept of limited slip between the slab and beam. The typical ⅞-in. dia. 4-in. stud con-

USEFULL CAPACITY OF ONE STUD CONNECTOR IN POUNDS

STUD DIA. IN.	CONCRETE STRENGTH PSI			
	2500	3000	3500	4000
5/8	6,500	7,100	7,600	8,200
3/4	9,300	10,200	11,000	11,800
7/8	12,700	13,900	15,000	16,100

$$\text{DESIGN LOAD} = \frac{\text{USEFULL CAPACITY}}{\text{SAFETY FACTOR}}$$

SAFETY FACTOR RANGE FOR BUILDINGS = 2.4 to 3.5

NOTE: ABOVE TABLE DEVELOPED BY STUD CONNECTOR MANUFACTURE FROM RESEARCH DATA BY DR. I. M. VIEST FORMERLY RESEARCH PROF. UNIVERSITY OF ILLINOIS

CONNECTOR	ALLOWABLE HOR. SHEAR LOAD (Q) (KIPS) (APPL. ONLY TO CON. MADE WITH ASTM C33 AGGREGATES		
	F'_c = 3,000	F'_c = 3,500	F'_c = 4,000
$\frac{1}{2}$" DIAM. X 2" HOOKED OR HEADED STUD	5.1	5.5	5.9
$\frac{5}{8}$" DIAM. X 2.5" HOOKED OR HEADED STUD	8.0	8.6	9.2
$\frac{3}{4}$" DIAM X 3" HOOKED OR HEADED STUD	11.5	12.5	13.3
$\frac{7}{8}$" DIAM. X 3.5" HOOKED OR HEADED STUD	15.6	16.8	18.0

TABLE 1.11.4 (A.I.S.C. - 1961)

Fig. 8. Tables of shear connector capacities.

nectors used for the composite design on Cobo Hall used the shear values indicated in Fig. 8, with a safety factor of 2.5. This included allowance for the higher *n*-value of lightweight concrete.

The composite design was originally evaluated on the basis of cost of temporary shoring for Hall **C** to obtain composite action for dead as well as live load. This cost evaluation on such a system of high shores proved to be a false economy and the design was developed for composite action for live load only.

Construction Technique for the Composite Design with Lightweight Concrete—Lightweight concrete was used to great advantage on this project in many areas other than the composite design for Hall **C** and the framing over the expressway. Over 20,000 cu yd of lightweight concrete, using a coarse and fine aggregate of expanded shale processed by the rotary kiln method, maintained a maximum density for this type of concrete between 90 and 100 psf. This one-third reduction in dead weight for the long spans reduced the cost of steel framing as well as the cost of foundations far beyond the premium cost of using lightweight aggregate concrete instead of regular weight concrete.

In general, the design compressive strength specified for the lightweight concrete used in the composite design was 3750 psi at 28 days. To provide quality control and maximum performance in the lightweight concrete, extensive laboratory and field testing controls were specified. Full size trial mixes with the accepted aggregates and developed mix design were made before the concrete construction started.

The quality control in the field included not only making cylinders, but also checking plastic weight, slumps, air content, and field yield with each set of cylinders cast for every 75 cu yd of concrete placed. Quality control of concrete also included prewetting the aggregate stockpiles for moisture control and uniformity of placing of concrete.

To limit the amount of shrinkage as well as the possible loss of composite action between the floor slab and the steel beams and girders, the sequence of placing the concrete slab as well as the size of pours was limited by specification. The concrete was placed in sections starting in the center of the span first and working toward the supports. Slab sections over the main girders were left out and placed last, as shown in Fig. 9.

These controls contributed considerably to the quality and the performance of the lightweight concrete slabs. The quality control of the concrete as indicated by the cylinder tests shown in Fig. 10, with a coefficient of variation of 11 percent, indicated concrete of a quality better than expected.

U. S. POST OFFICE, DETROIT

Composite design using lightweight aggregate with structural steel was employed economically and successfully in Detroit's automated U. S. Post Office (Fig. 2). This facility, one of the largest of its kind in the nation, exemplifies the new concept of automated mail and parcel post handling. The huge new 23 million dollar project consists of a five-story reinforced concrete mail-handling building, a ten-story steel frame administration tower, and a double-deck truck maneuvering area the full length of the facility along the south side.

Fig. 9. Cobo Hall, Hall B, over John C. Lodge Expressway. Note lightweight concrete being placed in sections.

South Maneuvering Area—(See Figs. 11 to 14). The double-deck truck maneuvering area on the south side of the facility provides 45 truck spaces on the ground floor and 60 truck spaces on the supported deck above at the first floor level. The upper level is provided with truck ramps from the street.

The 90 to 95-ft spans required for the second level trucking area are designed for a 200 psf live load or 6 lanes of 20-ton truck loading (H20-S16-44), whichever produces the critical loading. These long spans and heavy loads are comparable to bridge design. Economy and the limited structural depth dictated the use of a composite construction of long span welded steel plate girders with shear connectors and a lightweight structural concrete deck.

Composite Design Technique with Lightweight Concrete—The final design consisted of welded plate girders with sloped top flanges, with a maximum depth of 5 ft-8 in. at the center of the span and 5 ft-0 in. at the supports. The long span girders spaced at 7 ft-6 in. on centers were made of ⅜-in. web plate and unbalanced flanges, consisting of a top flange 15 in. x 1 in. thick and a bottom flange 18 in. x 1½ in. thick, to provide an ideal and economical steel cross section for composite design.

The sloped top flange from the center of the long span girders provided the grade needed in the lightweight concrete deck for drainage and held the slab to the minimum thickness required for structural strength.

The composite design approach and construction technique for the South Maneuvering Area was similar to that adopted on Cobo Hall. This design approach followed the AASHO standards and available publications on composite design with modified criteria for the use of lightweight concrete in place of normal weight concrete.

The composite section, consisting of the welded plate girders and the lightweight concrete deck, was designed assuming a modular ratio n of 16. This ratio was higher than that used in the design of Cobo Hall due to the dynamic loading possible from truck traffic. The shear connectors were generally ⅞-in. dia. x 4 in. spaced according to the intensity of the horizontal shear along the span with the smallest spacing toward the supports of the plate girders. The useful capacity of the stud connectors

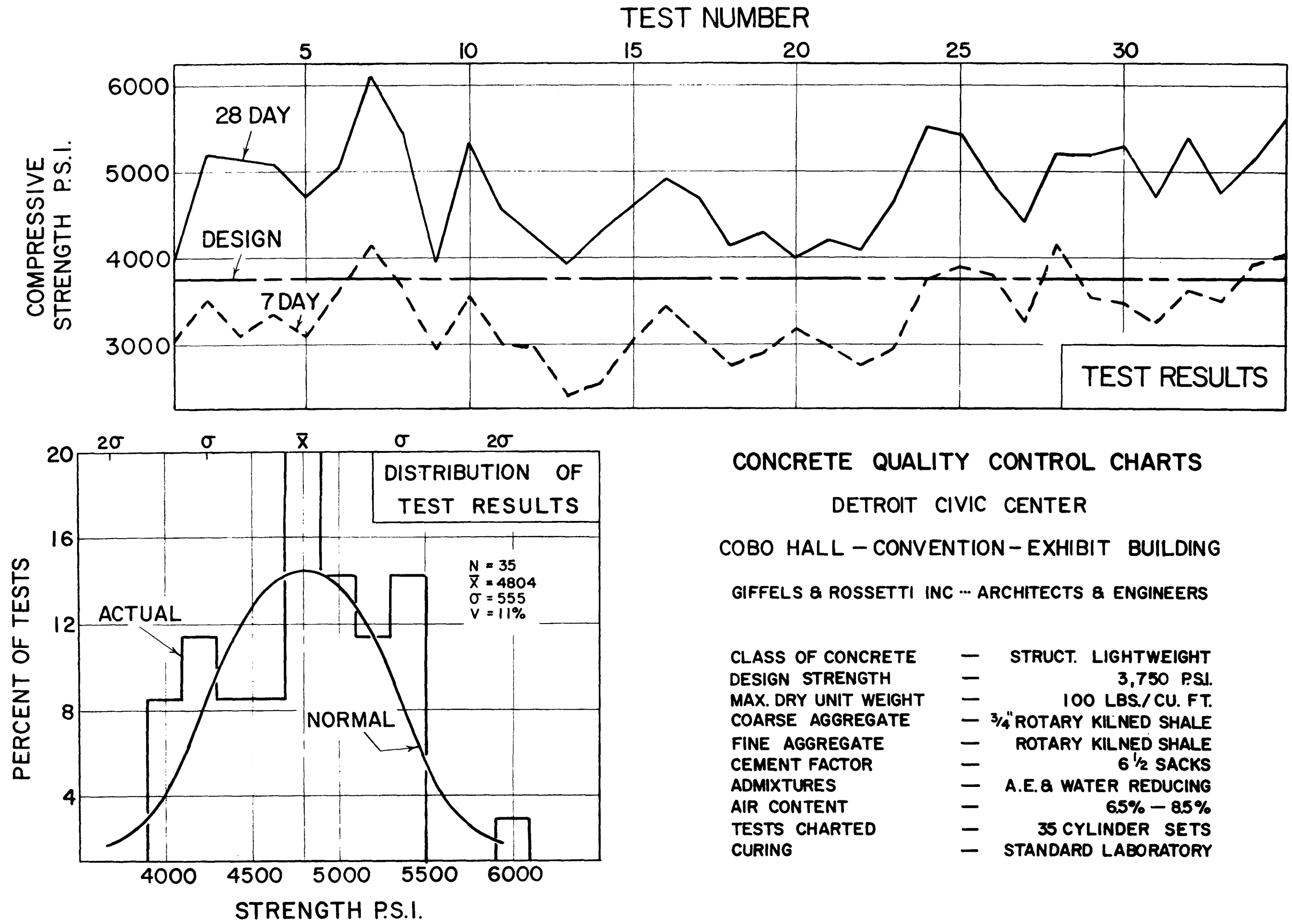

Fig. 10. Lightweight concrete quality control charts, Cobo Hall.

used in the design was approximately 5000 lbs per stud connector as determined from the table (see Fig. 8) with an assumed safety factor of 3. This safety factor also included an allowance for the dynamic loading possible on this type of facility.

Construction Technique with Composite Design—In the erection of the structural steel, the stud shear connectors were welded to the girders in the shop at the option of the steel fabricator (Fig. 12).

To minimize shrinkage and horizontal shear between the lightweight concrete deck and the top flange of the plate girders, the concrete was specified to be placed in alternate longitudinal strips 120 ft in maximum length and 20 ft in maximum width. This required the contractor to start in the middle of the steel spans and work outward towards the outer supports. Shoring of the steel girders during the placing of the lightweight concrete slab to utilize the effect of composite design under dead load as well as live load was not used on this project, due to the high cost of shoring as compared to the savings that could be made in the composite section.

In general, the use of lightweight concrete in composite design nullifies the economy gained by shoring for increased capacity of the composite system, except for very low cost shoring systems.

The lightweight concrete was made from expanded shale aggregate, rotary kiln processed, with a strength of 3750 psi and a maximum density of 105 pcf. The lightweight concrete on this facility allowed the use of natural sands for fine aggregate, proportioned not to exceed the maximum density. The use of natural sand in the concrete mix reduced shrinkage, improved the placing and control of the concrete, and reduced the premium cost of this type of concrete over normal weight concrete.

Quality control of the lightweight concrete was also a must on this project and consisted of the usual concrete cylinders as well as slump, plastic weight, air content, and field yield controls. The quality control charts as indicated in Fig. 13, with a coefficient of variation of 10 percent, are adjudged excellent by ACI standards for field-controlled concrete. The controlled quality concrete resulted in a minimum of nuisance problems generally associated with concrete framing of this type during and after construction.

COMPOSITE DESIGN OF COBO HALL AND U. S. POST OFFICE AS COMPARED TO PRESENT CODES AND PRACTICES

Although composite design using lightweight concrete in both Cobo Hall and U. S. Post Office was conservative by today's design standards, this type of construction has proved to be economical and highly successful in performance and service. Greater economy, however, can be realized in this type of construction by the latest revisions to such specifications as those of the AISC and ACI.

For instance, the AISC permits the uniform spacing of shear connectors between the points of contraflexure of the positive moment zones, which is more economical than the variable spacing of shear connectors according to the intensity of shear diagram. The higher allowable shear capacity of the connectors in normal weight concrete also indicates possible higher capacity values of shear connectors in lightweight concrete than was used on the projects described. The higher stress steels available today also indicate economy when used in combination with composite design, especially with lightweight concrete. Therefore, it appears that today's acceptable standards of composite design, if further developed,

Fig. 11. U. S. Post Office, view looking west. South maneuvering area on the left.

Fig. 12. U. S. Post Office, south maneuvering area, erection of welded plate girders. Note shear connectors shop welded on top flange.

would lead towards more economy than was realized in the original design of Cobo Hall and U. S. Post Office and should make this type of construction very competitive, especially for large spans.

EXISTING SPECIFICATIONS AS RELATED TO COMPOSITE DESIGN WITH LIGHTWEIGHT CONCRETE

The revised AISC Specification of 1961 and the ACI Code of 1963 introduced design standards for composite design with normal weight concrete. These specification revisions, as mentioned at the beginning of this paper, did not include the use of lightweight concrete in composite design.

The AISC Specification revisions were specifically written around the use of normal weight aggregates as defined by ASTM C33 with the composite section computed in accordance with the elastic theory and the shear connector requirements based on the ultimate strength concept.

The revisions to the ACI Code with regard to composite design primarily deal with precast or cast-in-place concrete and exclude any reference to the use of composite design with cast-in-place concrete slabs and steel beams or girders. Furthermore, the ACI Code recognizes composite concrete construction on the basis of working stress design as well as ultimate strength design only for an all-concrete section. The concrete code has developed the composite standards of design along a concept rather remote from those followed by the AISC Specification. However, the 1963 ACI Code revisions did include and recognize the structural characteristics of lightweight concrete and define these characteristics with regard to modulus of elasticity, working stress, etc. as related to normal weight concrete, but not as related to composite design.

As a result of these revisions in the AISC and ACI Standards, a gap still exists with regard to the application of lightweight concrete in composite design. For instance, a clear-cut standard for the shear capacity of shear connectors used in composite design with lightweight concrete is not given by the AISC. Consequently, practicing engineers have used assumed values, based on past experience, that have ranged anywhere from 60 to 85 percent, and in some cases, 100 percent of the capacity

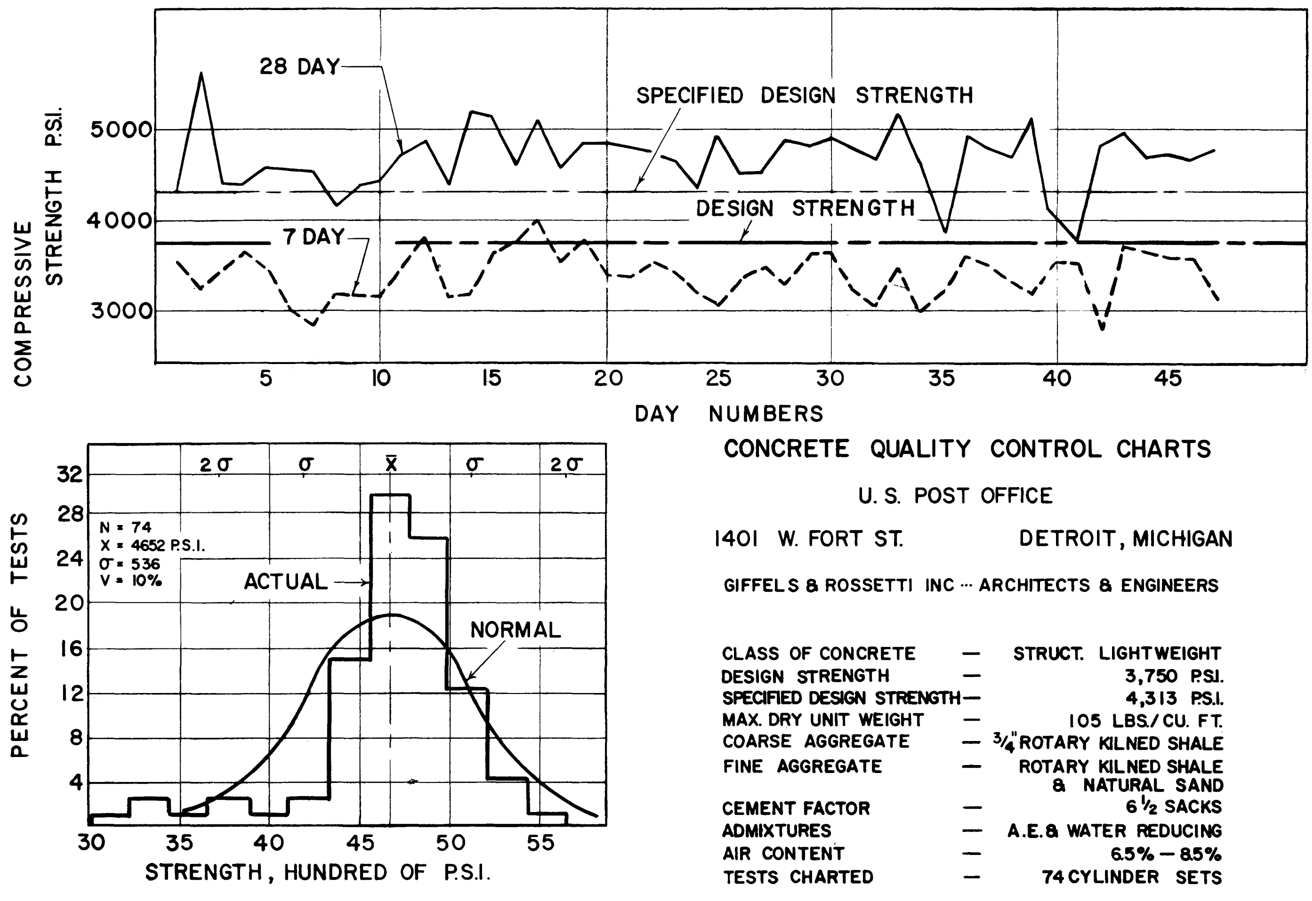

Fig. 13. Lightweight concrete quality control charts, U. S. Post Office.

Fig. 14. U. S. Post Office, south maneuvering area, view looking east.

of shear connectors for normal weight concrete, as given in Table 1.11.4 of Section 1.11 (see Fig. 8) of the AISC Specifications.

The modular ratio n required for computing the transformed section of composite design, although clearly defined in the latest ACI revisions with regard to an all-concrete section, is not specific and does not cover its application to composite design with concrete and steel. As written, the ACI Code specifies that the modulus of elasticity E_c of lightweight concrete is to be used only for calculating deflections. The code therefore implies that the n-value in computing the transformed section for composite design would be the same as normal weight concrete.

This assumption, used by some engineers, has been contested by others and leaves some doubt and controversy, since the section modulus can vary as much as 15 percent by using the n-value of regular weight concrete instead of the n-value of lightweight concrete. In large spans, this percentage difference could be critical.

As a result of the lack of design information that exists in the area of composite design with lightweight concrete as defined by the latest additions to the AISC and ACI specifications, the author feels that there is a need for research and development as well as code revisions to clarify and define design criteria for the use of composite design with lightweight concrete.

Items such as allowable horizontal shear load for connectors used with lightweight concrete, and the modular ratio n as related to composite design with lightweight concrete, should be investigated and defined.

It should be recognized that the economy of composite construction and the competitive position of this type of framing system is enhanced by the use of lightweight concrete and the reduced dead load as clearly indicated by the experience and success of this construction on Cobo Hall and the Detroit U. S. Post Office.