A Stub-Girder System for High-Rise Buildings

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THE CURRENT TREND to "systems" design in building construction has necessitated a new interdependence between the owner, architect, structural engineer, mechanical and electrical engineer, and the contractor. This paper describes one of the new systems—the integration of the mechanical ducts into the structural steel floor beams of a high-rise building. This system is called a *stub-girder* system.

A conventional system of framing a floor in a structural steel structure is shown in Fig. 1(a). The structural system consists of wide flange beams spaced between 7 ft-0 in. and 11 ft-0 in. apart. The floor slab consists of approximately 3¼ in. of lightweight concrete on a metal deck. Composite action between the steel beams and the concrete slab is generally achieved by the use of suitable shear connectors. The mechanical ducts, lights, and ceiling construction are generally placed under the beams. In some instances, penetrations are made in the beams and girders to accommodate the ducts. For a building having a span of 40 ft-0 in. between the core and the exterior columns, the distance between the top of the floor slab and the ceiling is approximately 4 ft-0 in.

The stub-girder system is shown in Fig. 1(b). The system consists of a girder spaced approximately 30 ft-0 in. on centers and spanning between the core and an exterior column. This girder consists of a high-strength, wide flange beam with stub pieces shop-welded on the top flange. Floor beams are placed over the girder (between the stub pieces) at approximately 10 ft-0 in. on centers. The floor beams are designed for continuity and are spliced near the points of inflection. The system is completed by the placement of a lightweight concrete slab on metal deck spanning between the floor beams. Composite action is ensured by the provision of shear connectors. The girder is shored while the concrete reaches its design strength.

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LOAD TEST OF STUB-GIRDER

A load test was conducted at the test facilities of Granco Steel Products Company in St. Louis. The details of the test girder are shown in Fig. 2. The girder consisted of a W14×48 bottom section, W16×26 stub pieces and floor beams, and a 3000 psi lightweight concrete slab having a thickness of 3¼ in. above the 2-in. Cofar deck. Shear connectors were placed to connect the stub pieces to the concrete slab. The girder was shored until the concrete developed its design strength. On the day of the test, the concrete strength was 3178 psi. The width of the slab was 5 ft-0 in. due to testing limitations. Also, since A572 Gr. 50 steel specified for the W14×48 girder was not available, A36 steel was used. The load was applied as concentrated loads at the three floor beam locations to simulate as closely as possible the actual behavior of the floor. Strain indicators were placed at several locations both on the bottom flange of the girder as well as the top of the concrete slab. Deflection dials were placed at the location of the middle floor beam which was almost at the center of the test beam.

The load was applied in six increments to the total load of 28.3 kips per load point. (This load is the design load for the actual structure, but due to the testing limitations listed above, the equivalent design load for the test beam was 20.4 kips per load point.) The load was applied and then removed three times to study the composite action between the stub pieces and the concrete slab. The load was then reapplied to failure.

The load-deflection curve at first loading is shown in Fig. 3. The curve is essentially linear. At a load of 28.3 kips the deflection was 1.15 in. (span/405).

The maximum steel strains in the bottom flange at the center load point are shown in Fig. 4. The maximum measured steel strain at the bottom flange of the girder

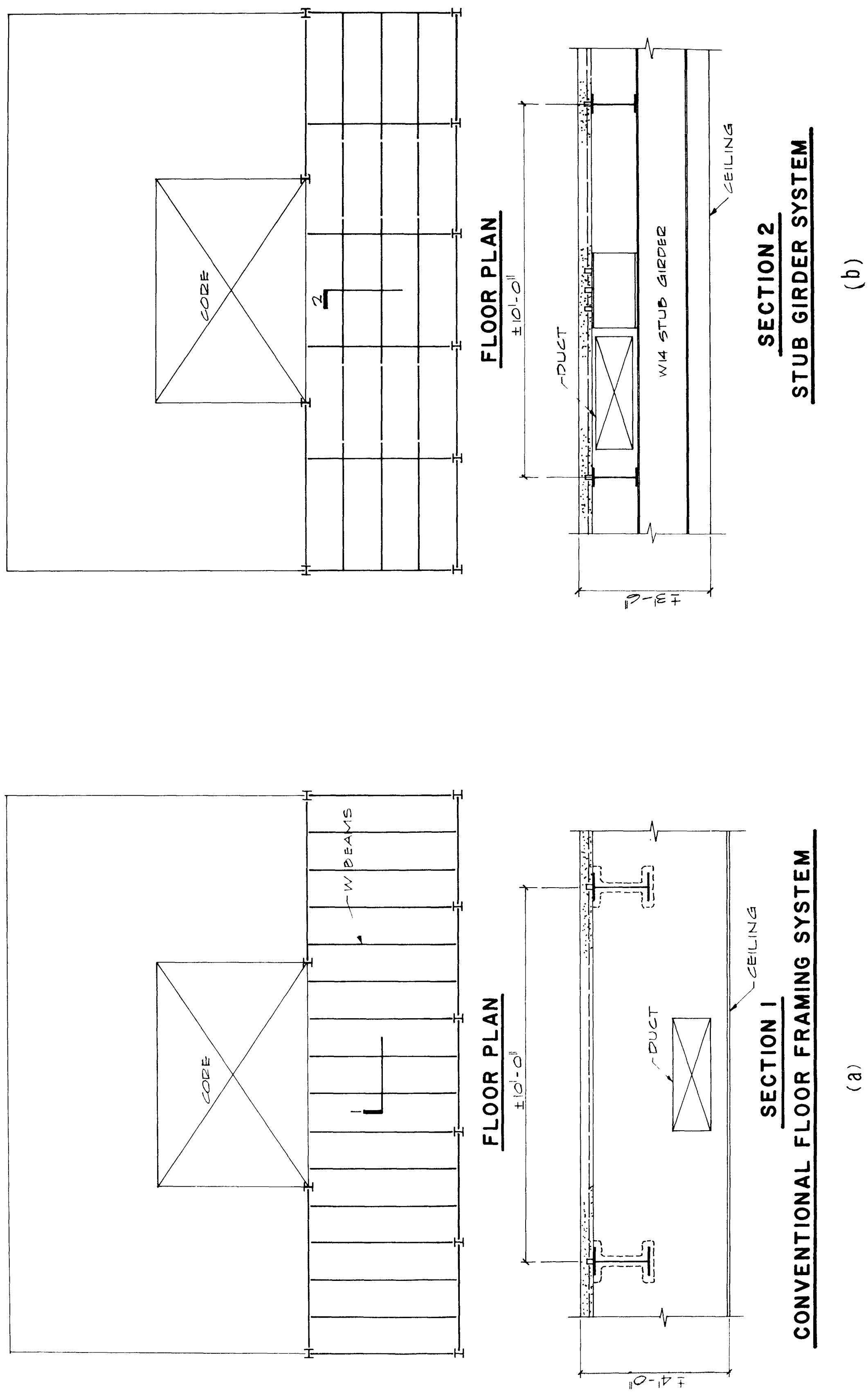

Fig. 1. Floor framing systems

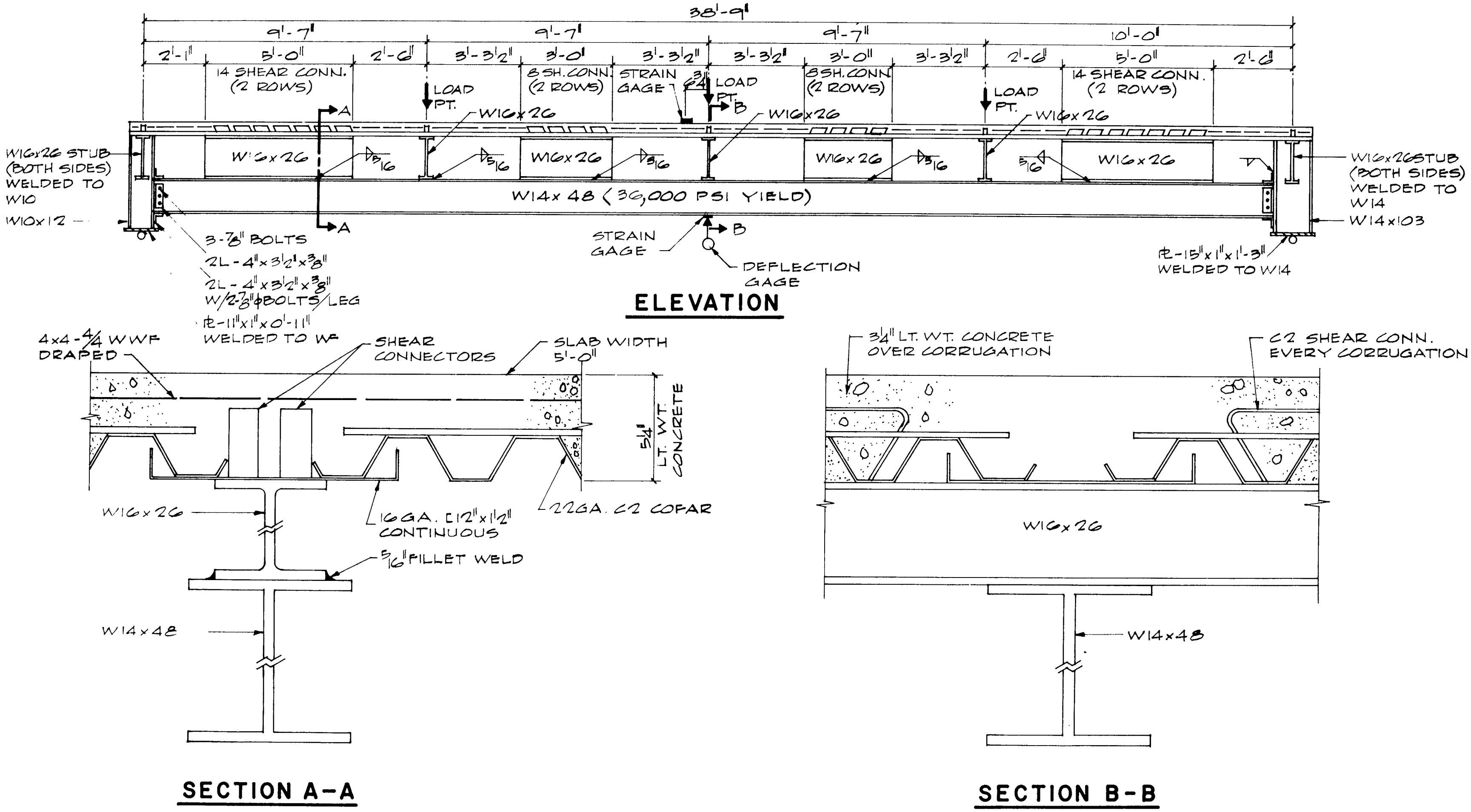

Fig. 2. Stub-girder load test

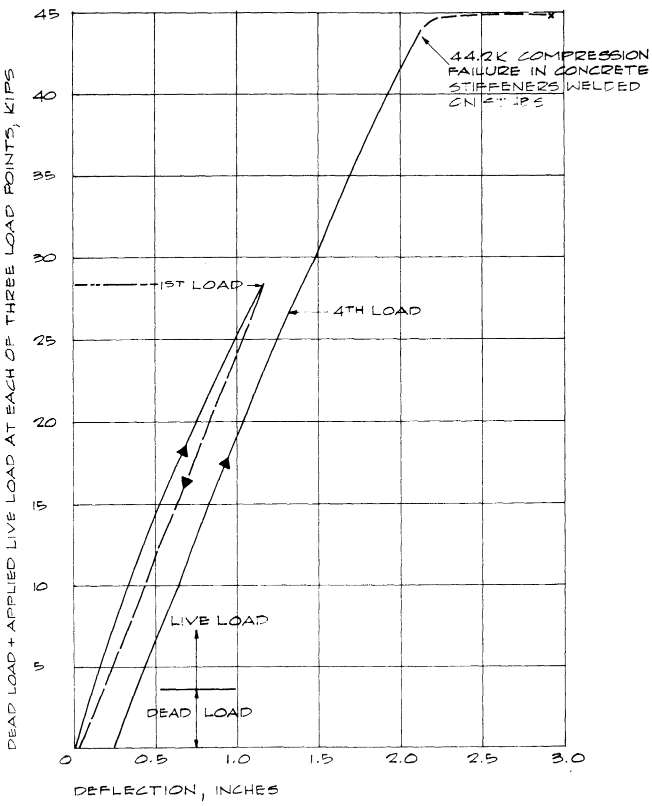

Fig. 3. Deflection under center load point

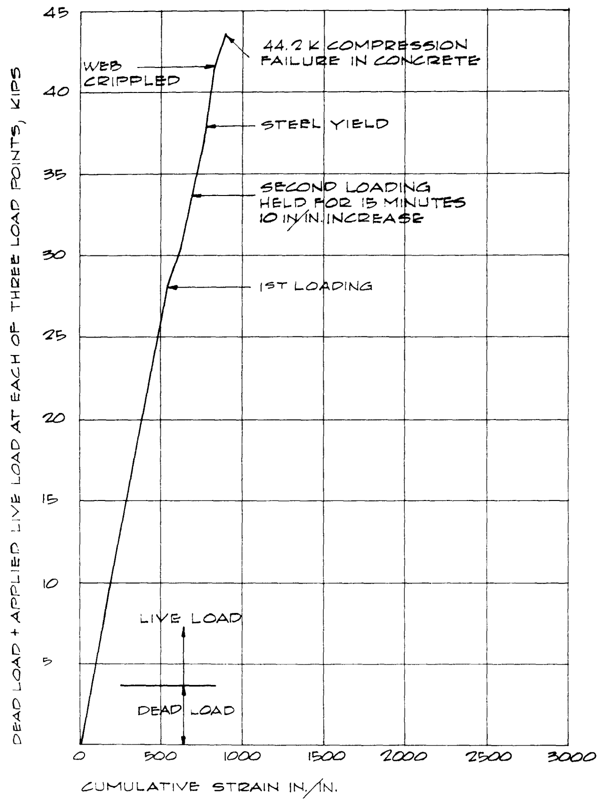

Fig. 5. Strain on slab surface, 6 in. from center load point

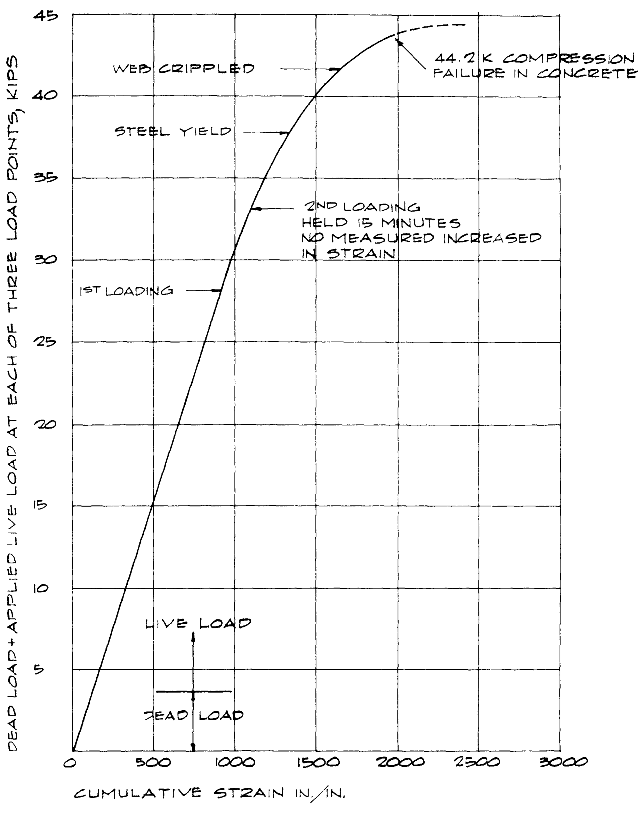

Fig. 4. Strain at bottom flange at center load point

at total design load was 940 micro-in./in., which corresponds to a steel stress of 27,300 psi. The strains in the top slab surface 6 in. from the center load point are shown in Fig. 5. The strain in the concrete slab at design load was 560 micro-in./in., which corresponds to a concrete stress of 1170 psi.

On the fourth load cycle the girder was tested to failure. The steel strains on the bottom flange reached the yield point at a total load of approximately 37 kips. At a load of 38.2 kips per load point the web crippled on the exterior end of the exterior stub piece. Inspection of the web indicated that the steel web was delaminated. Subsequent loading failed to show a similar behavior at the other end of the girder. Further loading increased the strains and deflections and at a total load of 44.2 kips, the concrete slab crushed at the edge of the first stub piece approximately 7 ft-0 in. from the support. Stiffeners were welded on the web of the stub and the load test continued. On reloading, a maximum load of 45 kips was reached at a deflection of 2.8 in. This results in a factor of safety of 2.2 on the equivalent design load. An examination of 'the specimen failed to disclose a separation between the concrete slab and the stub pieces, indicating that composite behavior was maintained throughout the test.

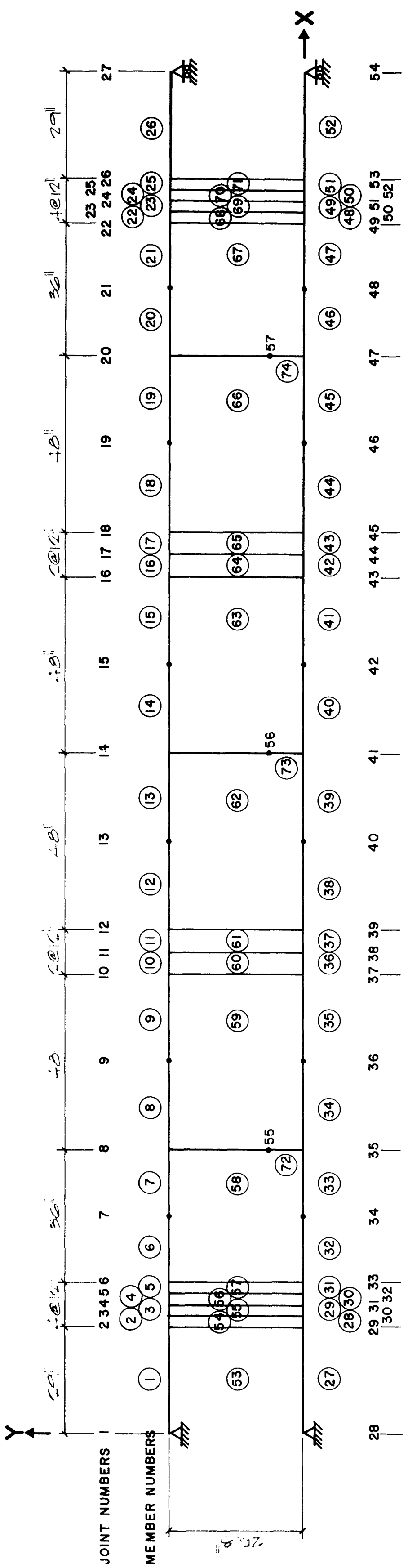

Fig. 6. Stub-girder analysis as Vierendeel

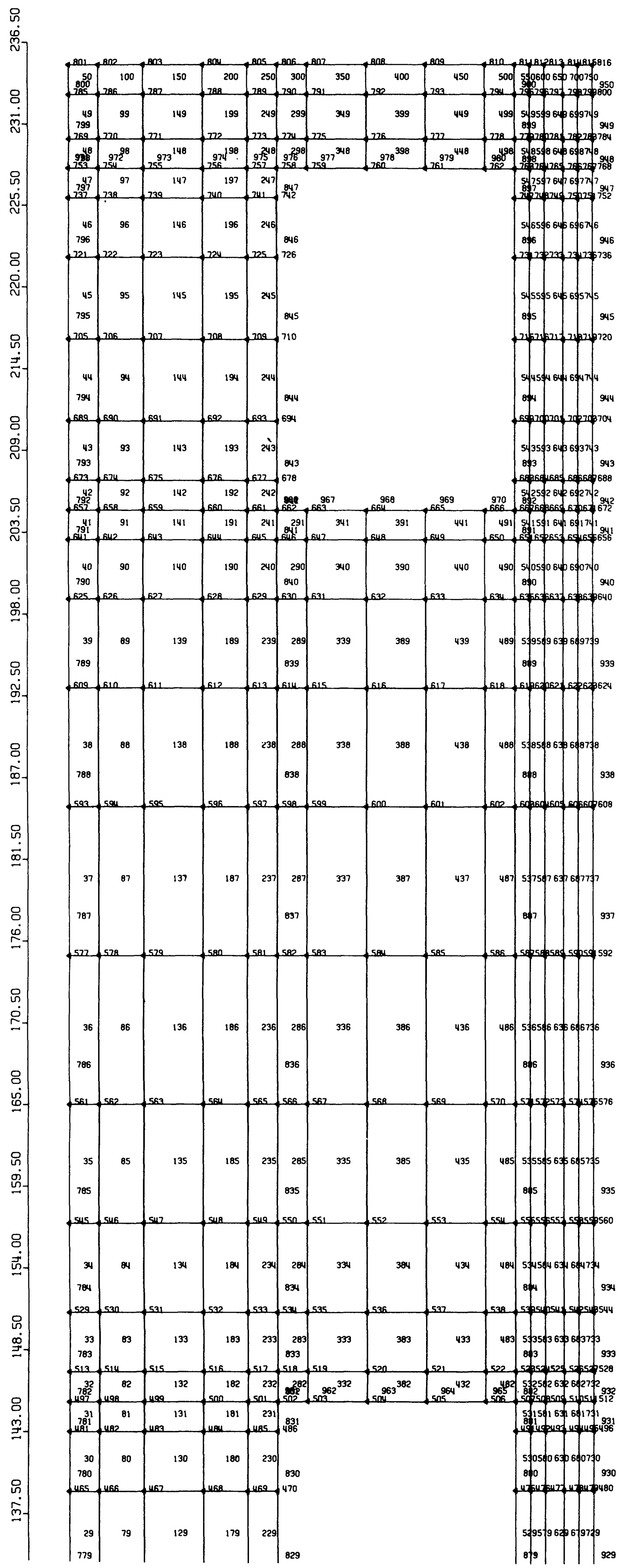

Fig. 7. Portion of finite element model

ANALYSIS OF STUB-GIRDER SYSTEM

Preliminary analyses of the girder were made as a non-prismatic beam and as a Vierendeel beam with the stub pieces simulated as verticals. A model of the Vierendeel beam is shown in Fig. 6. A more refined analysis was then conducted using a finite element analysis. A sketch of the model is shown in Fig. 7. The deflections, concrete stress and steel stress distributions obtained by the three analyses are compared with measured values in Fig. 8. It should be noted that the maximum concrete and steel stresses do not occur at the same fiber over the length of the beam. Consequently, the curves reflect the maximum stress envelopes.

A very close correlation was obtained between the analytical results and the measured values at midspan, as can be seen in Fig. 8. In general, the Vierendeel beam and finite element analysis give a better picture of the behavior of the girder, since the secondary moments are considered. The Vierendeel beam analysis agreed better with the maximum measured steel stress, while the finite element analysis gave a better agreement with the maximum measured concrete stress. Both the Vierendeel analysis and the finite element analysis indicated high concrete stresses at the edge of the first stub piece where the crushing of the concrete caused final failure.

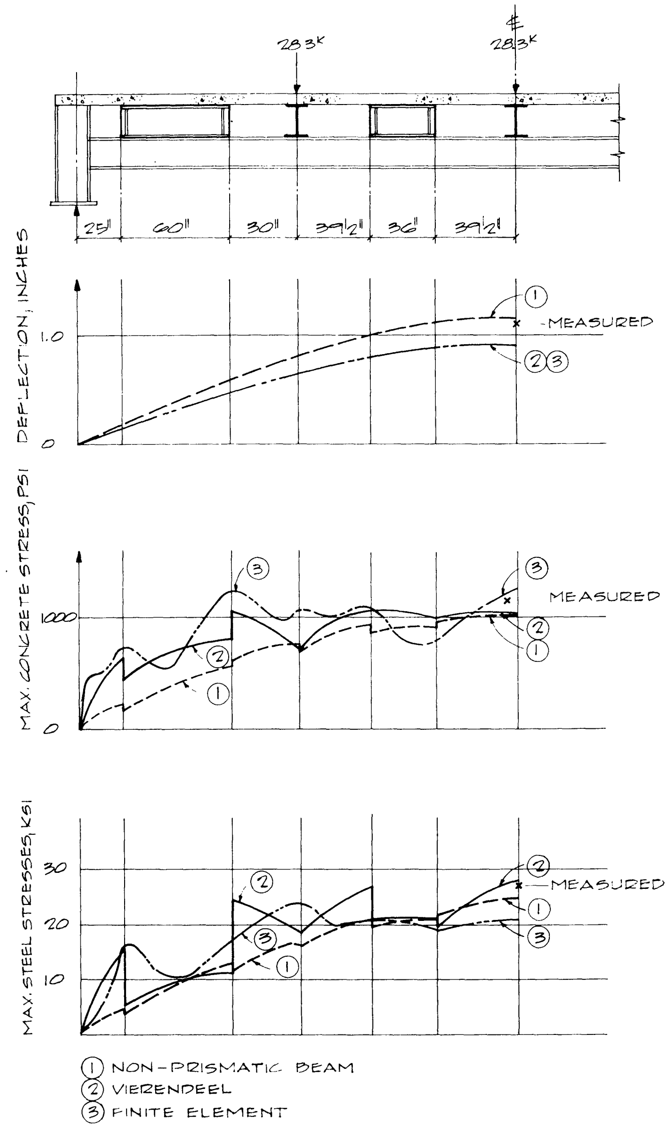

Fig. 8. Comparison of analytical results with test measurements

ADVANTAGES OF THE STUB-GIRDER SYSTEM

1. A reduction in steel required in the girder due to the greater depth.

2. Reduction in steel in the floor beams due to continuity. There is also a simplification of the end connection details of the floor beams due to lower shear values.

3. There is an overall reduction of approximately 25 percent in the structural steel in the floor and approximately 15 percent in the structural cost of the floor system.

4. There is a reduction of approximately 8 in. in the total depth between the top of the slab and the ceiling. This results in a lower floor-to-floor height and additional savings in the exterior window wall system for the building.

PRACTICAL APPLICATION

The stub-girder was first used on One Allen Center, a 34-story office building in downtown Houston. This building, the first in a 21-acre project being developed by Trammell Crow, was completed in November, 1971. The Architect is Wilson, Morris, Crain and Anderson.

The building measures 218-ft x 128-ft in floor plan with exterior columns spaced at 30 ft-0 in. centers. The distance from the exterior glass line to the core is 40 ft-0 in.

Several structural steel floor systems were priced on the project. Price comparisons indicated that the stub-girder system was substantially more economical than the conventional floor systems.

The stub-girder system used on the project was similar to the test girder in all respects except for the addition of longitudinal reinforcing bars in the slab to improve the ductility of the system.

Steel erection was started on November 1, 1970. Figure 9 shows the erection of the first tier of structural steel which includes the stub-girders. Figure 10 gives an underside view of the system after the placement of the 22-gage C-2 Cofar galvanized metal deck. In Fig. 11 a view is shown of the floor system after the welding of the shear connectors. The reinforcing steel for the floor slab was placed and 3¼ in. of lightweight concrete was placed over the deck while the girder was shored. Structural steel construction progressed at a rapid pace and the structural steel was topped out in October, 1971. The lateral stiffness is obtained by a composite exterior frame consisting of steel erection columns and a cast-in-place concrete frame. Precast concrete panels were used for the exterior facade of the building and also to form the concrete columns and spandrels as shown in Fig. 12. The total structure including the exterior panels was completed 12 months after the start of steel erection.

ACKNOWLEDGMENT

The writer would like to acknowledge the generous contribution of the staff of Granco Steel Products Company in the load test and Dr. Bill Gunnin of Ellisor & Tanner, Inc., Dallas, Texas, in programming the finite element analysis.

Figure 9

Figure 10

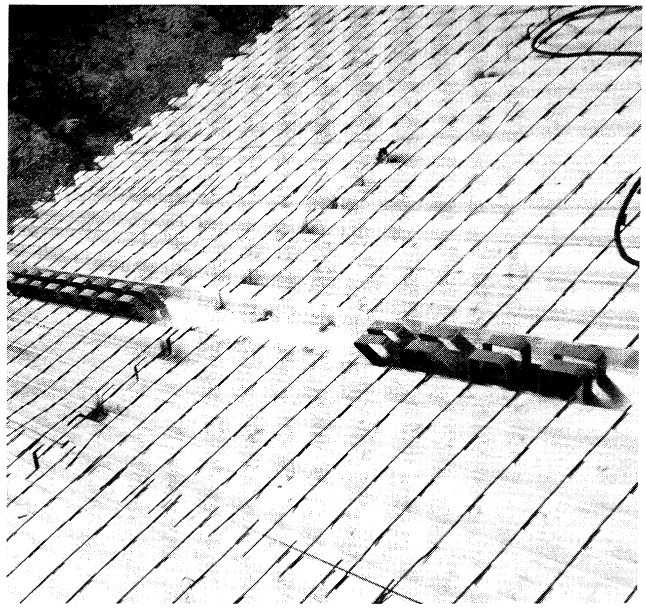

Figure 11

Figure 12