Composite Floor System for Sears Tower

S. H. IYENGAR AND J. J. ZILS

STRUCTURAL FLOOR SYSTEMS in high-rise steel buildings have usually taken the form of composite or non-composite rolled beam stringers which are spaced at 8 ft to 10 ft centers. In most cases, these stringers frame into moment-connected girders which are part of the traditional plane-frame system. The span of the stringer is governed by the economically feasible frame span and generally ranges from 20 ft to 40 ft. Therefore, the economic potential of floor systems beyond the 40 ft span for high-rise buildings has been inhibited to a large extent by the inefficiency of rigid frames over long spans in resisting lateral loads. Also contributing to the inefficiency is the indirect mode of load transfer from stringer to frame grider to columns. The design procedures for the most part have centered on determination of an acceptable frame span on the basis of required lateral resistance; the stringer system is then filled into this framework. The floor stringer subsystem has, therefore, played a secondary role in overall system optimization.

Recent developments of different types of structural systems, most notably "tubular" structures, have lifted the restrictions that were imposed by frame buildings. In an exterior tube system where all lateral stiffness is derived from the perimeter framing, the floor stringers span from exterior columns to building core columns. For optimum design of the exterior structure of the equivalent tube, it is desirable to transmit a large proportion of the total gravity load to the perimeter columns. This is achieved by the use of longspan stringers. Spans in the range of 45 ft to 60 ft have been used for these structures. Figure 1 shows an arrangement of floor framing members for a typical exterior framed-tube system. It can be seen that the indirect mode of load transfer from stringer-to-girder-to-column still exists for the interior of the building, most notably in the corner areas of the building. In terms of optimum development of lateral stiffness, a possible inefficiency of the exterior framed-tube system is caused by the nonparticipation of the core columns in sharing the lateral load and presence of large shear lag.

S. H. Iyengar is Associate Partner, Skidmore, Owings & Merrill, Chicago, Illinois

J. J. Zils is Project Engineer, Skidmore, Owings & Merrill, Chicago, Illinois

A geometric restructuring of the tubular lines of the exterior framed tube to improve cantilever efficiency (by reduction of shear lag effect) and to induce effective participation of core columns led to the development of the development of the "Bundled Tube" system (Fig. 2). The use of a long span floor system in the span range of 70 ft to 80 ft was essential to this geometric reformulation. The structure for the world's tallest building, Sears Tower, currently under construction in Chicago, is based on a Bundled Tube system. Essentially, the system is composed of nine square modular tubes of 75 ft dimension to formulate an overall Bundled Tube of 225 ft square, as shown in Fig. 2. Each tube terminates at a different height to suit the interior floor space requirements. Figure 3 shows a construction condition up to the 72nd floor of the Sears Tower. Two primary factors controlled the size of the modular tubes. One related to cantilever efficiency of the overall system and the other to efficiency of the floor framing system within each tube. Studies on modular tubes of different sizes to produce reasonably high cantilever efficiency resulted

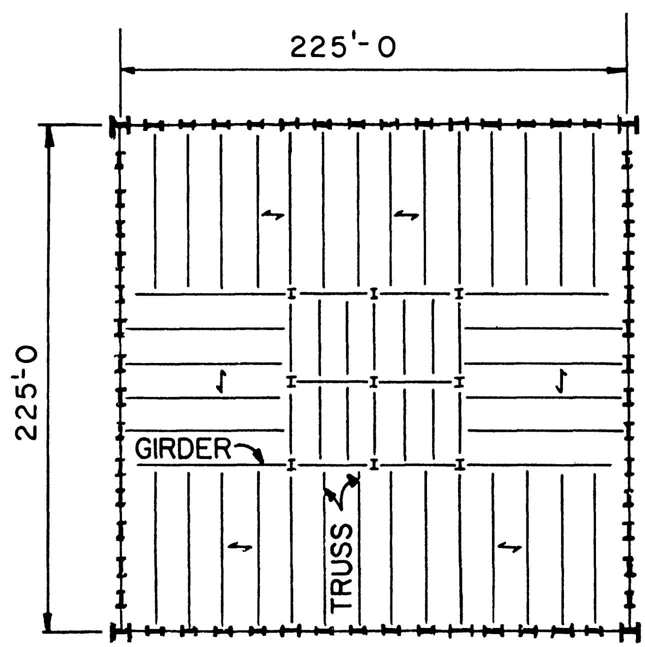

Fig. 1. Exterior frame "Tube"

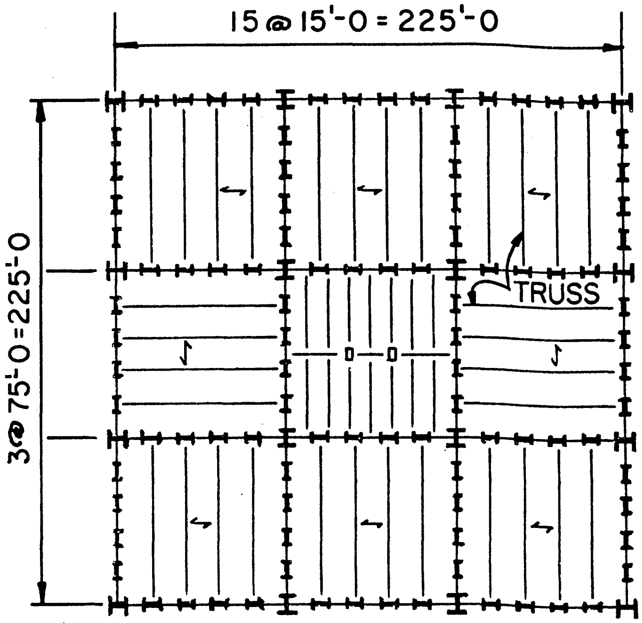

Fig. 2. Bundled framed "Tube"

in an optimum range between 60 ft and 90 ft. Such long spans in an ultra high-rise building would normally involve a considerable structural steel premium. However, because of the development of a highly efficient composite floor system, spans up to 80 ft were economically feasible.

Composite design for the floor members was instrumental in floor steel reductions on the order of 2 lb/sq ft. Further, the larger bending stiffness was beneficial in reducing floor vibrations. The positive mechanical tie between the slab, floor members, and space-frame girders developed an effective floor diaphragm. The diaphragm rigidly ties all modular tubes together by virtue of its large in-plane shear stiffness and aids in lateral shear distribution between different frames of the Bundled Tube.

Normally, composite action is developed with a solid concrete slab. However, the recent trend has been to use metal deck systems as a composite slab to reduce labor costs. This, together with a 3-in. deep cellular system for power distribution, alters some of the basic characteristics of the normal composite behavior. A systematic experimental study was undertaken to verify the composite behavior. The study included extensive push-off tests to determinine shear stud strengths, a beam test to establish correlation between push-off tests and actual beam behavior, and a full scale truss test to verify the stiffness behavior of the floor system. Other tests included composite slab tests and verification of vibration characteristics of the floor. This paper describes the results of the experimental study and consideration for the optimum design of the overall floor system.

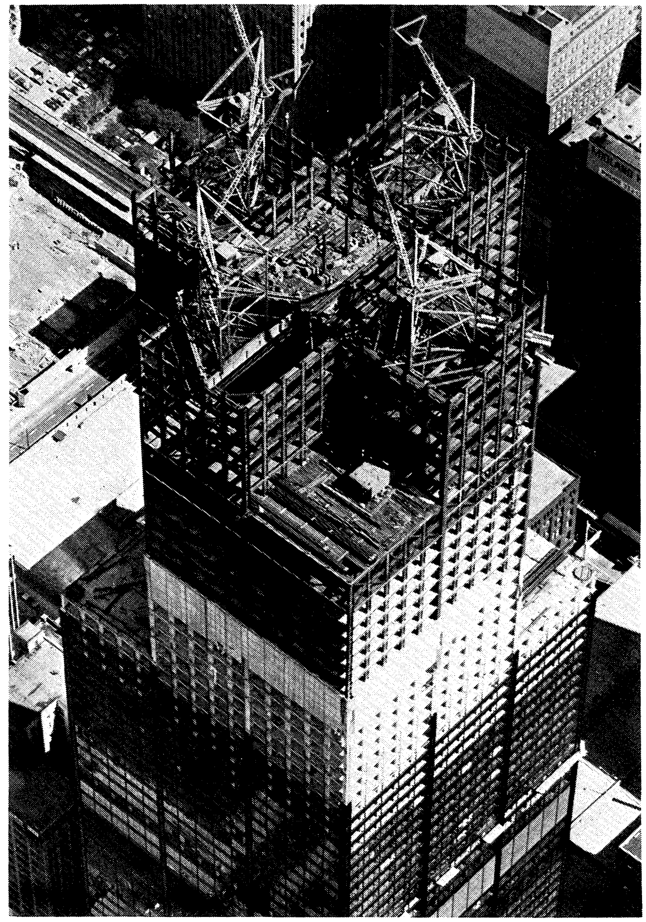

Fig. 3. Tower construction to 72nd floor

SELECTION OF FRAMING SYSTEM

Even though the square shape of the modular tube is ideal for two-way framing, large fabrication and erection premiums are involved with this type of steel framing system because of extensive field welding at rigid joints and the large number of pieces to be erected. A one-way framing system was, therefore, selected over a two-way system. Rolled steel beams are generally used for floor members which normally result in an economic framing system. However, for the 75-ft span, the deepest available rolled member of 36 in. would have allowed inadequate space below the beam for the passage of mechanical ducts, thus requiring either a larger floor-to-floor height or beam web penetrations. A significant premium results in either case. For an optimum solution, a system which utilizes all the available depth in the floor-ceiling sandwich space, and whose natural configuration allows for passage of ducts and pipes, was required. Castellated beams can meet the depth criteria, but the extent of the web opening was not adequate for mechanical distribution. The logical solution was a truss configuration with shallow chord depths to maxi-

Fig. 4. Passage of mechanical ducts through truss

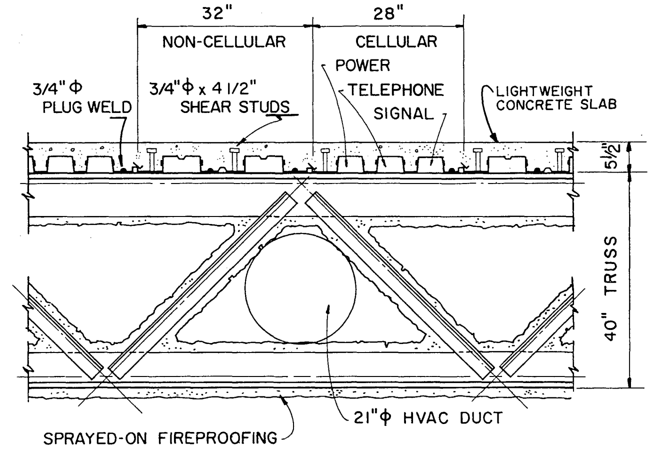

Fig. 5. Composite truss assembly

mize clear opening space between the chords as shown in Fig. 4. Since a large number of trusses were required, they were mass produced in a jig without any significant premium. The use of simple joint details without gusset plates was essential in minimizing this premium.

DESCRIPTION OF THE FLOOR SYSTEM

The floor system typically consists of 75-ft Warren type trusses at 15-ft centers, as shown in Fig. 2. Each truss is connected directly to a column by means of high strength bolts designed for shear only and is, therefore, considered simply supported. The circuitous load transfer from stringer to girder to column, which is evident in other systems, has been totally eliminated in the Bundled Tube system. The trusses are 40 in. deep and their design was based on composite action with the floor slab. The trusses occupy all the clear depth in the floor-ceiling space, which resulted in a normal floor-to-floor height of 12 ft-10 in. The floor slab consists of a composite assembly of 3-in. blended metal deck and 2½-in. structural lightweight concrete topping for a total slab thickness of 5½ in. The composite deck spans the 15-ft distance between the trusses. The blend includes a 28-in. cellular portion for electrical and telephone services and a 32-in. non-cellular portion comprising a 5-ft module. The composite action for the truss was established by ¾-in. diameter x 4½-in. long shear studs welded through the metal deck in the noncellular portions. The composite assembly is shown in Fig. 5. Figure 6 is a construction photograph of the deck-truss system.

The span direction of the trusses located in the corner tubes was alternated over groups of six floors to equalize the loading on the tube walls. The lateral support for the top chord of the truss was established by the floor diaphragm and, therefore, no bridging between the trusses was required. The columns at each floor are laterally supported either by trusses or by a composite T-section framing into trusses. The mechanical tie between the slab and all floor members is provided by shear studs. Other floor members consist of secondary framing for core shaft openings.

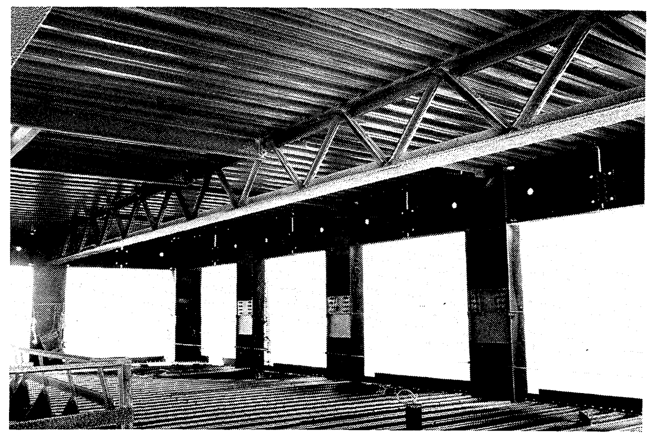

Fig. 6. Truss with metal deck

COMPOSITE BEHAVIOR

Steel beams which act compositely with the floor slab have been used quite extensively in building construction during the last decade. In earlier versions, wood formed and shored solid reinforced slabs were used as in bridge construction. More recently, several types of composite slab systems which use permanent metal deck as formwork have been developed. These systems have the advantage of reduced labor cost and speedier construction. The increased facility to weld shear studs through the metal deck and the use of this composite slab also to act compositely with the steel beam have been logical developments. In most existing applications, the slab spans generally range from 8 ft to 10 ft, utilizing a 1½-in. metal deck. In these systems, the electrical and telephone services are generally provided

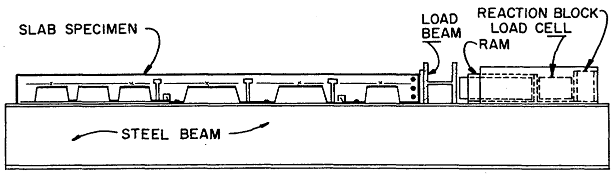

Fig. 7. Push-off specimen

by under-floor conduits which are punched through the slab at various locations. However, for the Sears Tower, the integration of the power distribution system as part of the structural slab imposed a minimum cell depth of 3 in. This deeper depth was beneficial in optimizing the deck design to support the wet weight of the concrete in an unshored condition for the longer than usual span of 15 ft.

Experimental verifications by J. Fisher[1] and others have indicated that there is very little loss of composite beam stiffness due to the ribbed configuration of the metal deck in the depth range up to 1½-in. In most of these cases, the entire strength of the shear stud can be developed, as in solid slabs, as long as the width-to-depth ratio of the troughs containing the shear studs is at least 1.75. However, with deeper metal decks, a substantial decrease in stud shear strength has been noted which can be attributed to a different mode of stud failure. The failure is initiated by cracking of the concrete in the rib corners and eventual failure takes place by separation of the concrete from the metal deck over a shear cone above the shear stud group. For these cases, the shear connector strength is closely related to the deck configuration and to all the factors related to the surface area of the shear cone.

The narrow troughs of the cellular deck are unsuitable for shear stud placement. Therefore, all shear studs are placed in the non-cellular part. The deck configuration in the non-cellular part was derived by the manufacturer (INRYCO) to respond to the optimum use of metal, the largest possible width-to-depth ratio of the trough, and other criteria related to the manufacture and erection of the deck. When subjected to lateral shear, the concrete above the metal deck tends to behave as a portal frame with the concrete in the troughs acting as rigid columns and concrete over the humps acting as beams. Increased shear strength is obtained with a stiffer portal frame. This means that the widest troughs and narrowest humps will yield the highest connector strengths. However, this trend increases the volume of concrete above the metal deck and decreases the section modulus of the deck alone and, therefore, requires thicker metal to carry the wet weight of the concrete slab. Some reasonable modulation of the trough and hump widths was required to optimize the overall cost. The resultant configuration of the deck can be seen in Fig. 5.

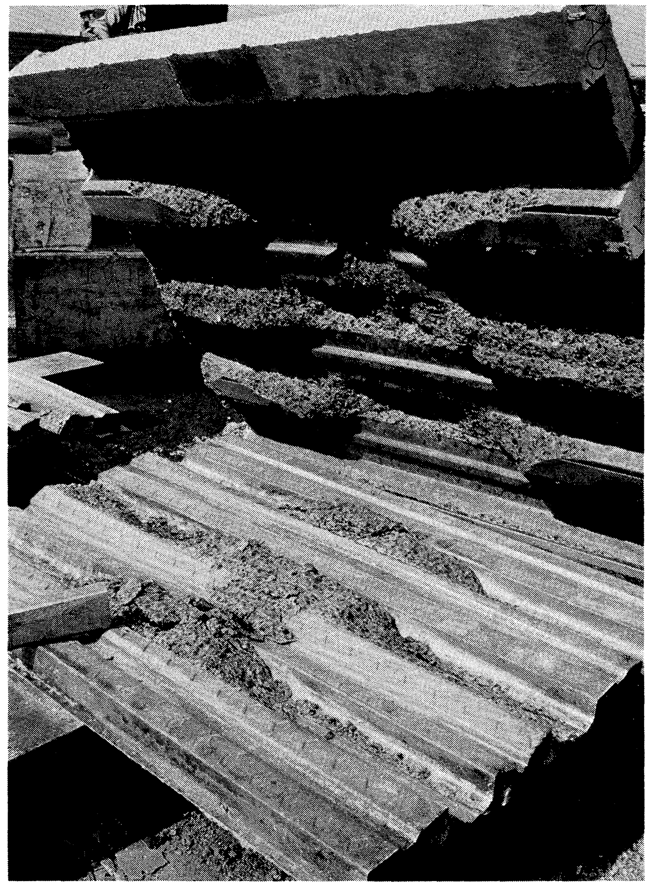

Fig. 8. Shear cone failure above shear stud groups

TESTING FOR COMPOSITE BEHAVIOR

A review of available test results on 3-in. deck systems of various configurations indicated the need for further testing with this deck configuration. The testing program was modeled to determine the shear stud capacities for different stud groupings, the effectiveness of plug welding in augmenting the shear strength, the effect of stud length, and factors related to spacing and location of studs. This was accomplished by a series of push-off tests and a beam test. The composite slab behavior was verified by single and double span tests. These tests were performed by INRYCO. The stiffness and vibration characteristics of the composite truss assembly were verified by a full-scale test performed by American Bridge Division. The results of these tests are discussed as follows:

Push-Off Tests—Figure 7 shows a typical arrangement of a push-off specimen. The width of the slab perpendicular to the load was typically 6 ft. All failures were in the slab over a shear cone as shown in Fig. 8. Tests were performed for the blended deck and for the non-cellular deck. The results are described with respect to each factor.

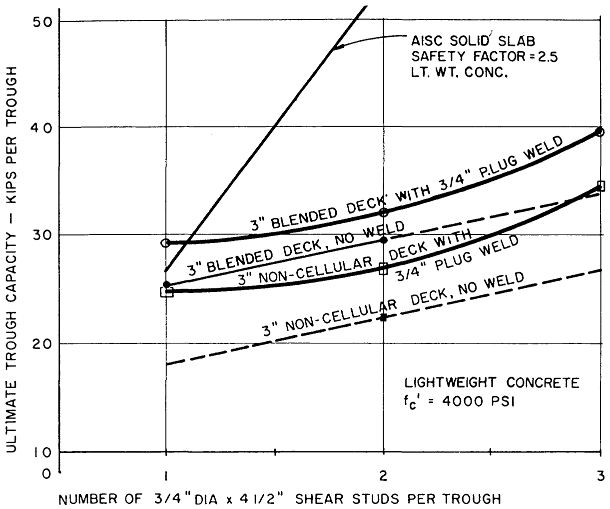

Fig. 9. Influence of metal deck and plug weld on deck trough capacity

The Effect of Stud Groups—When more than one stud is provided in a trough, the respective shear cones overlap; consequently, the shear value per stud is decreased.

The ultimate shear values obtained for 1, 2, 3 and 5 studs per trough are shown in Figs. 9, 10 and 11. Figure 9 shows the dramatic difference between the solid slab values obtained from the AISC Specification and various cases involving the metal deck. The divergence increases with the number of studs per trough. Figure 10 shows the results as a function of the ultimate stud shear value. The average stud values are 29.15, 16.0, and 13.2 kips for the 1, 2, and 3 stud cases, respectively, with blended deck and lightweight concrete. This represents a drop of 45 percent and 56 percent for the 2 and 3 stud cases as compared to the single stud case. The average shear stud values for the non-cellular deck with light weight concrete are 24.8, 13.50, and 11.5 kips representing similar percent reductions. It is interesting to note that the blended deck shows higher strengths compared to the non-cellular deck, which may be attributable to the stiffer equivalent concrete portal frame. This difference in strength is about 20 percent. This could also be observed in Fig. 9. Even though considerable reductions occur in stud values, some increase in trough capacity can be obtained by increasing the number of studs per trough. However, it is apparent that beyond a certain number of studs there may be no further increase in trough strength. The result of the 5 stud case for the non-cellular deck using regular weight concrete shows a reduction of 64 percent of the single stud value (Fig. 11.)

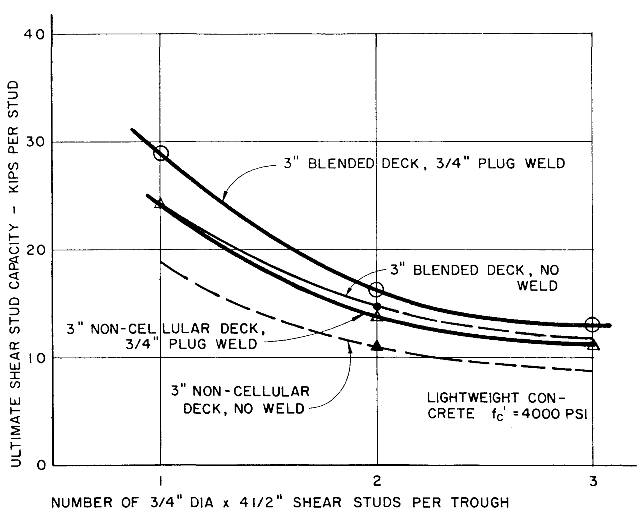

Fig. 10. Influence of metal deck and plug weld on shear stud capacity

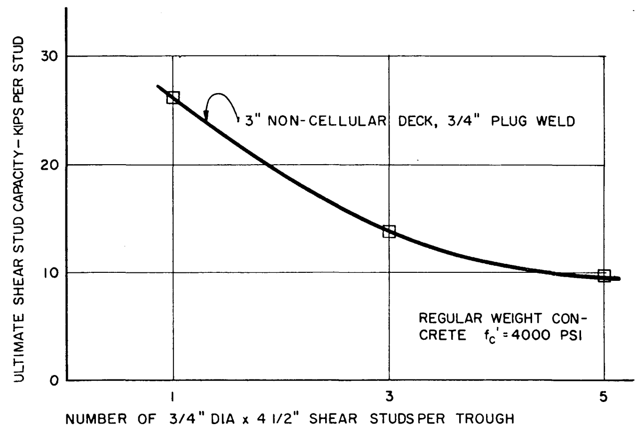

Fig. 11. Influence of metal deck and plug weld on shear stud capacity

Table 1 summarizes the ultimate shear loads used for design purposes for various stud groupings. A factor of 1.67 was applied to these loads to derive the allowable stud values to be used with AISC shear forces.

The Effect of Plug Welds—Some initial testing was performed without plug welds. Since the studs were located to one side of the trough, there was considerable rotation of the trough which appears to have initiated earlier cracking of the rib concrete. In later tests, plug welds were provided on the sides of the troughs opposite to the shear studs to uniformly anchor the trough down to the beam. This resulted in substantial increases in shear values, as can be observed from Fig. 9. It is possible that some strength may have been derived by direct shear resistance of the plug welds.

Table 1. Design Ultimate Shear Stud Capacities

BLENDED DECK LIGHTWEIGHT CONCRETE			NON-CELLULAR DECK LIGHTWEIGHT CONCRETE			NON-CELLULAR DECK REGULAR WEIGHT CONC		
STUDS PER TROUGH			STUDS PER TROUGH			STUDS PER TROUGH		
1	2	3	1	2	3	1	3	5
29.15	16.00	13.20	24.80	13.50	11.50	26.22	13.75	9.45

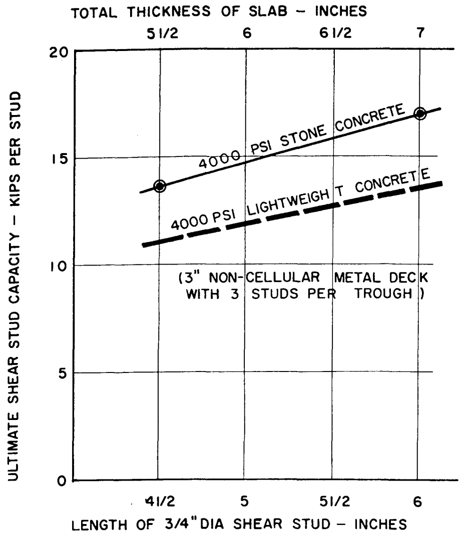

Fig. 12. Shear stud capacity versus length of shear stud

Stud Location in Trough—Results of the push-off tests have indicated that shear studs located on the side of the trough toward the beam support are more effective than studs located toward the beam center line side of the trough. Since the concrete pushes the studs away from the midspan point of the beam, a larger volume of concrete between the stud and the pushing side of the trough appears to prevent penetration of this stud into the opposite side of the trough.

Length of Shear Stud—The length of the shear stud has a definite effect on the shear stud value. As the length of the shear stud increases, so does the size of the shear cone. Consequently, the shear stud value is increased. Results of push-off tests for 4½-in. long and 6-in. long studs in stone concrete are shown in Fig. 12. The values of 13.5 kips/stud and 17.0 kips/stud, respectively, for the 4½-in. and 6-in. studs indicate an average increase of approximately 1.2 kips/stud for each additional ½-in. of stud projection above the 3-in. deck. Figure 12 also shows an equivalent curve derived for lightweight concrete based on $\sqrt{E_{c-i}/E_{c-n}}$ ratio, where E_{c-i} and E_{c-n} are elastic moduli of lightweight and normal weight concretes. This curve results in an average increase of approximately 0.95 kips/stud for each additional ½-in. of shear stud projection.

Push-Out Tests Versus Push-Off Tests—In the push-out test, the slab elements are cast on both beam flanges and the beam is pushed out while the slabs are supported. In the push-off test, the slab element is cast only on one flange and the slab is pushed off from the beam. Previous comparative results of the push-out and push-off methods of testing have indicated larger values for the push-out test for the 1½-in. deck case. However, tests were performed with the 3-in. blended deck for comparative evaluation. The results indicated close correlation with only insignificant increases for the push-out tests.

Beam Test—A composite beam test was modeled to substantiate the shear values obtained from the push-off tests. The test specimen was designed with only 50 percent shear connection and the test beam was larger than would normally be used for the test span of 38 ft. A 6-ft wide composite slab with 3-in. blended deck, which duplicated all elements and details of the typical slab, was cast on the beam. Two studs (¾ x 4½ in.) per trough were placed over the end one-third spans and single studs were placed in the middle third. Strain gages on the top and bottom surface of the beam were provided at various span locations. Six-point loading was used to simulate uniform loading. The mode of failure was by separation of the concrete slab from the deck with the characteristic shear cones over the stud groups. Synthesis of the test data at the inception of first cracking of a rib indicated an average value of 16 kips/stud with a maximum of 20 kips/stud at the location of the cracking. The corresponding push-off value was 16 kips/stud. The push-off method of testing, even though more severe, represents a valid method to verify the shear stud strengths. Figure 13 shows a plot of the stud shear distribution over a half span obtained from the beam test and the full scale truss test. Relative stud capacities over segments between strain gage locations have been plotted. Four segmental averages were available from the beam test and two from the truss test. Even though the data is insufficient to derive the shear values on each group of studs, the general trend is obvious. The slip produced by the rib and stud bending renders the shear distribution relatively uniform over the span length. The theoretical distribution for an infinitely rigid connection between the slab and beam is indicated by the dashed line. The loss of overall beam stiffness is about 12 percent as judged by the center span deflection at the load corresponding to first cracking. It was anticipated that the reduction of the stiffness under 100 percent shear connection would be smaller.

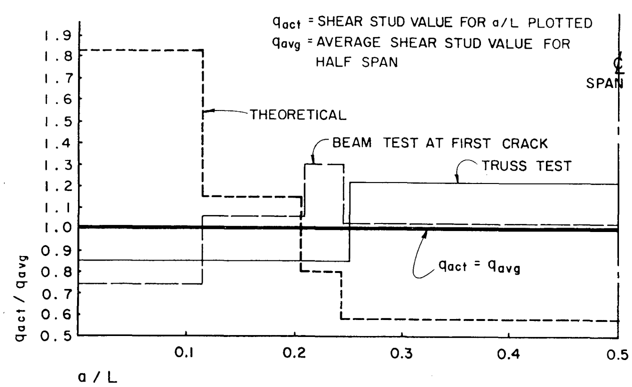

Fig. 13. Beam and truss shear stud capacities

Truss Test—The test assembly consisted of two 75-ft span trusses spaced at 15-ft centers. Spanning 15 ft between floor trusses and overhanging each truss by 5 ft was a 5½-in. deep composite slab with a 3-in. blended metal deck. A different type of deck with a 6-in. trough width was used because of the nonavailability of the exact type of deck finally used for the typical floors. This difference was considered insignificant for the purposes of the test. The trusses duplicated all features of the typical floor truss and were bolted to a relatively rigid bulkhead at both ends. Uniform loadings were applied over the entire span up to a superimposed load magnitude of 150 psf which with the weight of the system corresponds to 1.4 DL + 1.7 LL. Deflections and strains were measured at ¼-span points.

A comparison of the results of the truss test and theoretical analysis is shown in Table 2. The analysis was performed in two stages. For the dead load before hardening of the concrete, the truss was analyzed as a normal, non-composite truss. The depth of the truss corresponded to the distance between the center of gravity of the top and bottom chords and the diagonal lengths were measured to this center of gravity. An analysis was performed with a two dimensional frame program with all joints considered rigid. Support rotation was allowed to simulate simple support conditions. For the superimposed loads, composite properties were used for the top chord only. A modular ratio of $n = 9$ was used together with the 2½ in. of concrete above the deck.

A review of Table 2 indicates close correspondence of the deflections and stresses between the test and theory, which indicates the validity of the type of analysis performed. The members of the truss were designed from such an analysis using the regular provisions of the AISC Specification.

Figure 14 shows the load-deflection relationship obtained from the test which indicates practically full composite effectiveness. No cracks or failure were observed over the entire duration of the test.

Slab Tests—Typical blended and non-cellular decks were tested over a single-span condition and a two-span test was also performed for the typical blended deck

Table 2. Comparison of Truss Test Results versus Theoretical Analysis

		MID-SPAN DEFLECTION (INCHES)	BOTTOM CHORD STRESS (KSI)	TOP CHORD STRESS (KSI)	END TENSION DIAGONAL STRESS (KSI)	END COMPRSSN DIAGONAL STRESS (KSI)
STEEL TRUSS ONLY	ANALYSIS	1.00	5.13	6.28	5.15	3.90
	TEST	1.10	5.20	6.20	4.84	3.60
COMPOSITE TRUSS & CONC SLAB	ANALYSIS	3.65	28.53	10.00	29.65	22.65
	TEST	3.78	31.20	12.12	29.00	24.25

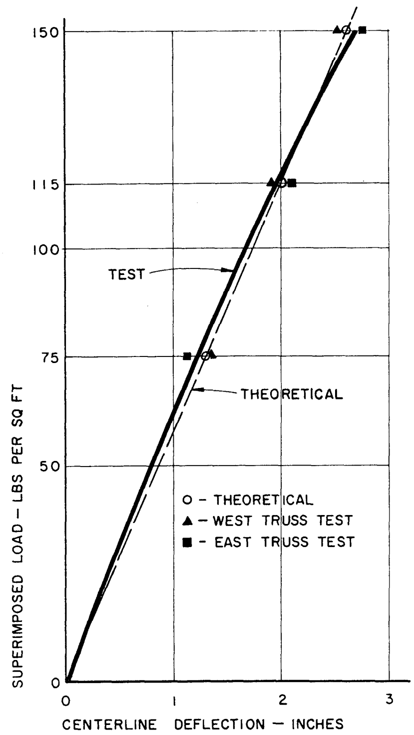

Fig. 14. Composite load-deflection curve-truss test

with the middle support continuous. The slab width was 5 ft and a four-point loading on each span was used to simulate uniform loads. The span ends were on rollers to simulate simple support.

For single spans, the actual stiffness and theoretically computed composite stiffness corresponded within a narrow margin up to the inception of yield in the deck, after which the deflection increased rapidly while the slab continued to carry increased loads. The failure was by shear bond and the ratio of the ultimate load to the load at the first yield was about 1.33. The deflection ratio of the ultimate to the yield was about 1.75. The ultimate to total working load ratio was about 3.8. Even though the shear bond failure was a sudden failure, the inception of yield at a load 25 percent less than the ultimate gave a reasonably acceptable ductile behavior before failure.

The two-span slab developed larger stiffness due to continuity up to a surprisingly large superimposed load even though only a nominal 6 x 6—10/10 mesh was provided in the slab. The first cracks over the middle support were observed at about 150 psf after which the stiffness and load behavior corresponded to that of a simply supported single span. It is interesting to note that 150 psf represents a substantially higher load than the total working load of 115 psf.

The deck design was performed on a continuous basis for the wet weight of the concrete plus a small construction load for an unshored condition using the usual allowable stresses. Stresses due to superimposed loads were computed on a simply supported basis using composite properties. To be consistent with the AISC composite beam design, stresses up to 0.76 F_y were allowed for the combined loading.

VIBRATION PERCEPTION

The AISC Specification provides only an indirect guide to control floor vibrations by limiting the span-to-depth ratios of floor members. Actually, perceptibility of transverse floor vibrations due to human habitation is closely related to fundamental natural frequency of the floor, damping for transverse vibrations and amplitude of motion. Even though an exact expression for perceptibility relating all three parameters is not currently available, some existing, approximate guides could be used. One such guide is a chart developed by K. H. Lenzen[2] where the frequency and midspan amplitudes are plotted for various scales of perceptibility. A simple approach was used with these charts. The data from several existing buildings with known vibration performance which has been acceptable was plotted on the chart to serve as a datum for comparison. Since the chart does not account for the different levels of damping, the selected floors consisted of similar structural materials, details and partitions to represent approximately the same level of damping as the particular floor being evaluated. The fundamental period of the composite truss system was computed at 4 sec. However, field verification on the actual floor with a portable accelerameter indicated this period to be about 5 sec. This is attributable to the larger stiffness obtained due to some end fixity of the friction bolted connection and slab orthotropic action. The comparative evaluation of the level of perception indicated general correspondence with existing buildings used in the comparison.

CONCLUSIONS

The authors have discussed in this paper the development of an optimum floor system for Sears Tower. While descriptions pertain to Sears Tower in particular, many considerations are applicable to any floor system. The development of composite action with a 3-in. cellular metal deck represents a logical progression in the overall development of composite floor systems. In most instances, such composite action results not only in savings in steel, but also in better vibration and diaphragm characteristics. The paper also points to the importance of coordination between the floor subsystem and the overall structural system for ultra high-rise structures. The effective integration of structural and mechanical subsystems in the same floor-ceiling sandwich space offers the potential of considerable economy.

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