A Composite Action Duct-Beam System

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THIS PAPER describes a new method of utilizing steel in building construction, based on a research program undertaken by Reid & Tarics, Architects-Engineers, for the American Iron and Steel Institute. Specifically the project was intended to generate a structural system to be bid competitively for the URBS (University Residential Building System) Project of the University of California.

It appears, however, that the field of application of this concept may have general interest. Basically, the concept is to take advantage of the proven inherent economic advantages of composite design and to combine it with a beam shape (unsymmetrical rectangle) which will allow the beam to serve as an air duct for either warm or cool air. The structural frame of the building thus becomes the primary network of horizontal air distribution, eliminating conventional ductwork in part or in whole. The system has application wherever a central system must be used for ventilation. It has the advantages of minimum steel weight and long span economy inherent in a composite steel structure, coupled with the obvious savings in ductwork and story height.

The design team, composed of the authors and Leo P. T. Destin and Sidney Hoover (Chief Mechanical Engineer and Project Architect, respectively, at Reid and Tarics) studied the various ways in which the duct-beam system could be used mechanically and architecturally. An important suggestion by our fire consultant, Joseph Yockers (former California State Fire Marshal), pointed the way towards eliminating fire-dampers from the duct-beam system. This is possible if the duct-beam discharges air vertically upward through the floor instead of horizontally as it is being done in conventional construction.

In a typical duct-beam application, the beam fireproofing would also serve as thermal insulation for the beam. Studies indicate that in most building situations no condensation would occur on the beam if air conditioning were used. The details of how the duct-beam system can be adapted for air handling on several projects is covered at some length later in this paper.

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Early in the project, it became obvious that a further refinement could be introduced that would make the system economically more attractive for a larger variety of span and load conditions. This development involves the use of high strength light-gage formed steel members in composite action. Since the AISC Specification rules for composite design specifically cover the use of rolled sections or plate girders only, it was necessary to develop and test design assumptions concerning light-gage material.

The first phase of the program concerned itself with the practicality of connecting light-gage steel with concrete to achieve composite action. The second phase consisted of a full scale load test of a light-gage composite beam, and the third investigated the effects of various holes on the web strength of box beams, as well as the behavior of beam-to-column connections.

TESTS OF COMPOSITE ACTION

Pushout Tests—The purpose of the pushout tests was to determine the strength of concrete embedded studs when welded to $\frac{3}{16}$-in. and 10 ga. material. Four specimens of $\frac{3}{16}$-in. and six specimens of 10 ga. were fabricated. Some experimentation with current and voltage on mockup pieces of plate was performed to obtain welds of good visual characteristics, and then studs were welded to the specimens.

After concrete strengths reached $f_c' = 4000$ psi, the specimens were load tested. All but two specimens failed by tearing of the base metal and rotation of the studs. The two specimens with $\frac{3}{8}$-in. dia. studs on 10 ga. failed by shearing at the base of the stud. The load slip curves are shown in Figs. 1 and 2. The test results are summarized in Table 1.

Table 2 compares the test results with published values for studs welded to plate equal in thickness to one-half the stud diameter, as per AISC. As can be seen from Fig. 3, the efficiencies of studs on light gage as compared with strengths obtainable from thicker base plates are on the order of 62 to 73 percent. It

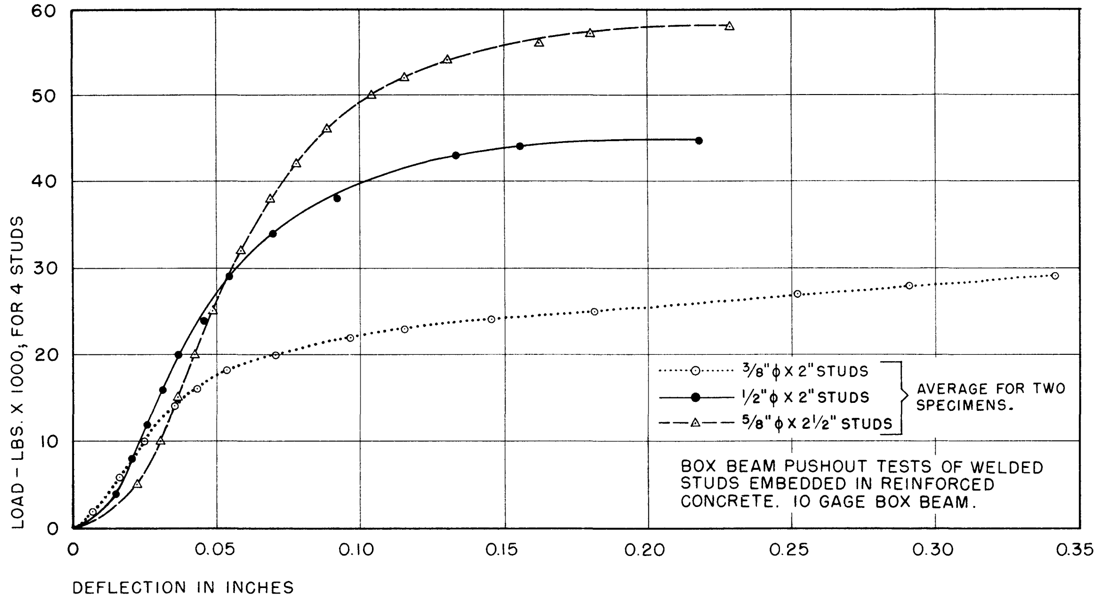

Fig. 1. Load slip curves for pushout tests

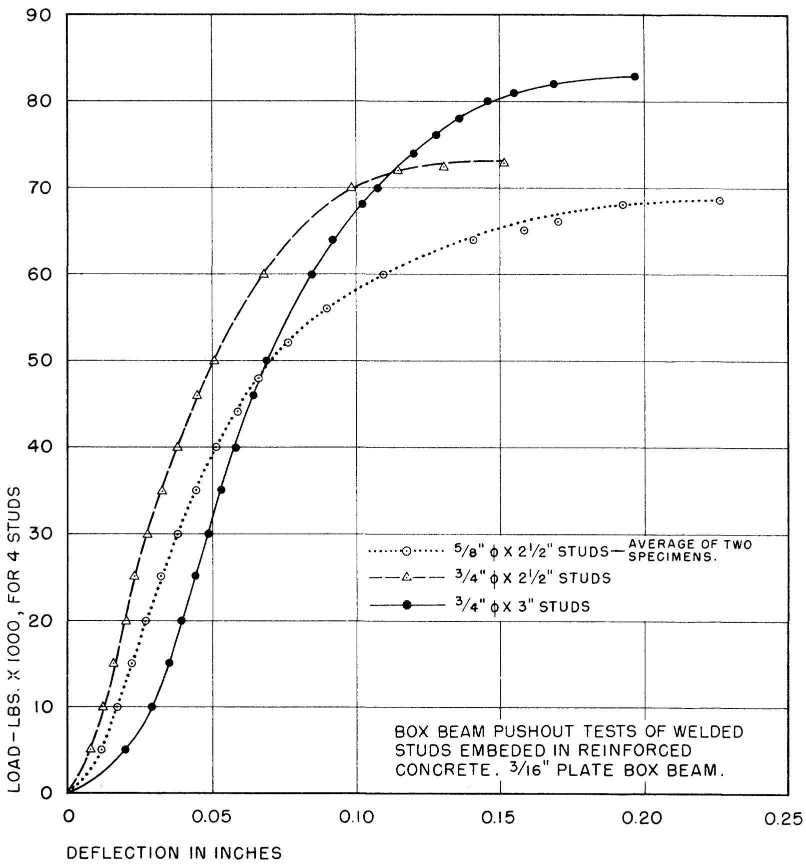

Fig. 2. Load slip curves for push out tests

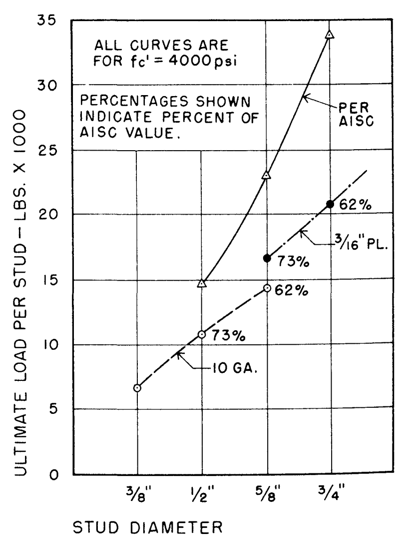

Fig. 3. Comparison of stud efficiencies

Table 1. Summary of Box Beam Pushout Tests

Item No.	Stud Size	Box Wall Thickness	Ultimate Load (4 Studs) lbs	Ultimate Load (1 Stud) lbs	Max. Slip in.	Failure Mode
1	3/8 x 2	10 ga.	29,000	7,250	0.342	Stud shear
2	3/8 x 2	10 ga.	25,150	6,287	0.203	Stud shear
3	1/2 x 2	10 ga.	44,800	11,200	0.219	[a]
4	1/2 x 2	10 ga.	41,000	10,250	0.178	[a]
5	5/8 x 2 1/2	10 ga.	56,200	14,050	0.177	[a]
6	5/8 x 2 1/2	10 ga.	57,800	14,450	0.229	[a]
7	5/8 x 2 1/2	3/16-in.	68,400	17,100	0.227	[a]
8	5/8 x 2 1/2	3/16-in.	65,200	16,300	0.162	[a]
9	3/4 x 3	3/16-in.	83,000	20,750	0.198	[a]
10	3/4 x 2 1/2	3/16-in.	73,000	18,250	0.152	[a]

$f_c' = 4{,}000$ psi

[a] *Spalling of concrete; bending of studs and tearing of parent metal around base of studs*

Table 2. Comparison of Ultimate Shear Capacity of Studs

Stud Size	q (ult) in kips as per AISC[a] (Min. t of conn. plate = 1/2 stud dia.)		q (ult) in kips as per test results[b]	
	$f_c' = 3{,}000$ psi	$f_c' = 4{,}000$ psi	10 ga. box bm. $f_c' = 4{,}000$ psi	3/16-in. plate box bm. $f_c' = 4{,}000$ psi
3/8 x 2	—	—	6.770	—
1/2 x 2	12.750	14.750	10.725	—
5/8 x 2 1/2	20.000	23.000	14.250	16.700
3/4 x 3	28.750	33.750	—	20.750
3/4 x 2 1/2	—	—	—	18.250

[a] *Figures shown in AISC Specification Table 1.11.4 are multiplied by 2.5 to obtain ultimate shear capacity of one stud (as per AISC).*
[b] *Test result values shown are average of ultimate shear capacity of two identical test specimens.*

appears, therefore, that economical connection of light gage beam to concrete is a definite possibility. Future investigations could conceivably find ways to achieve stud configurations of greater efficiency when welded to light gage material.

Load Test of a Full Size Light Gage Composite Duct-Beam—The beam was fabricated from 10 ga. steel with $F_y = 50$ ksi. Studs were welded to top flange (Fig. 4); one-half of the beam received 52 1/2-in. dia. x 2-in. studs, and the other half 40 5/8-in. dia. x 2.5-in. studs. The number of studs was determined by taking the pushout ultimate strength value and dividing by 2.5 to obtain safe working value. An air supply hole was cut in the top flange and the slab near the center of the span (see Fig. 5).

Figure 6 shows details of the test beam and the loading set up. Figure 7 shows the cross section of the beam and the properties of the section, as per AISC.

The following points are worth noting: The 10 ga. material and the bottom plate had a yield point of 50 ksi. A hypothetical working stress of 30 ksi was used to obtain an "allowable" moment of 219 kip-ft for the section. The allowable web strength, based on AISI rules, was computed to be 21.4 kips for the unstiffened light gage webs ($h/t = 14/0.1345 = 104$; v (allow.) = 5.9 ksi). When the concrete reached $f_c' = 4000$ psi,

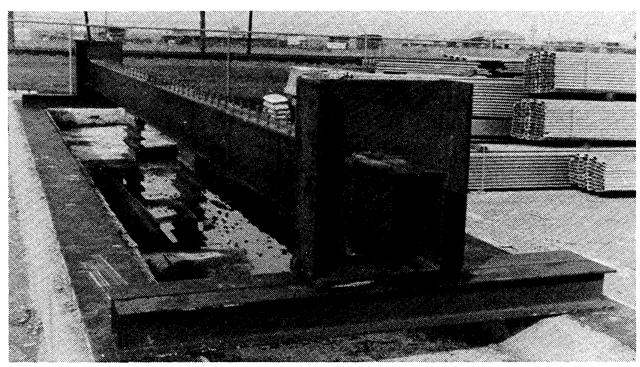

Fig. 4. Light gage duct-beam prior to concreting

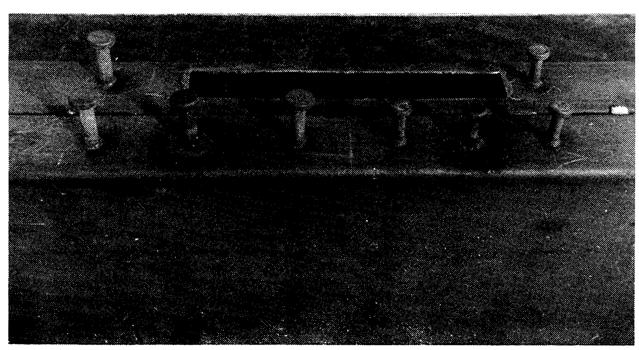

Fig. 5 Air supply hole in top of beam

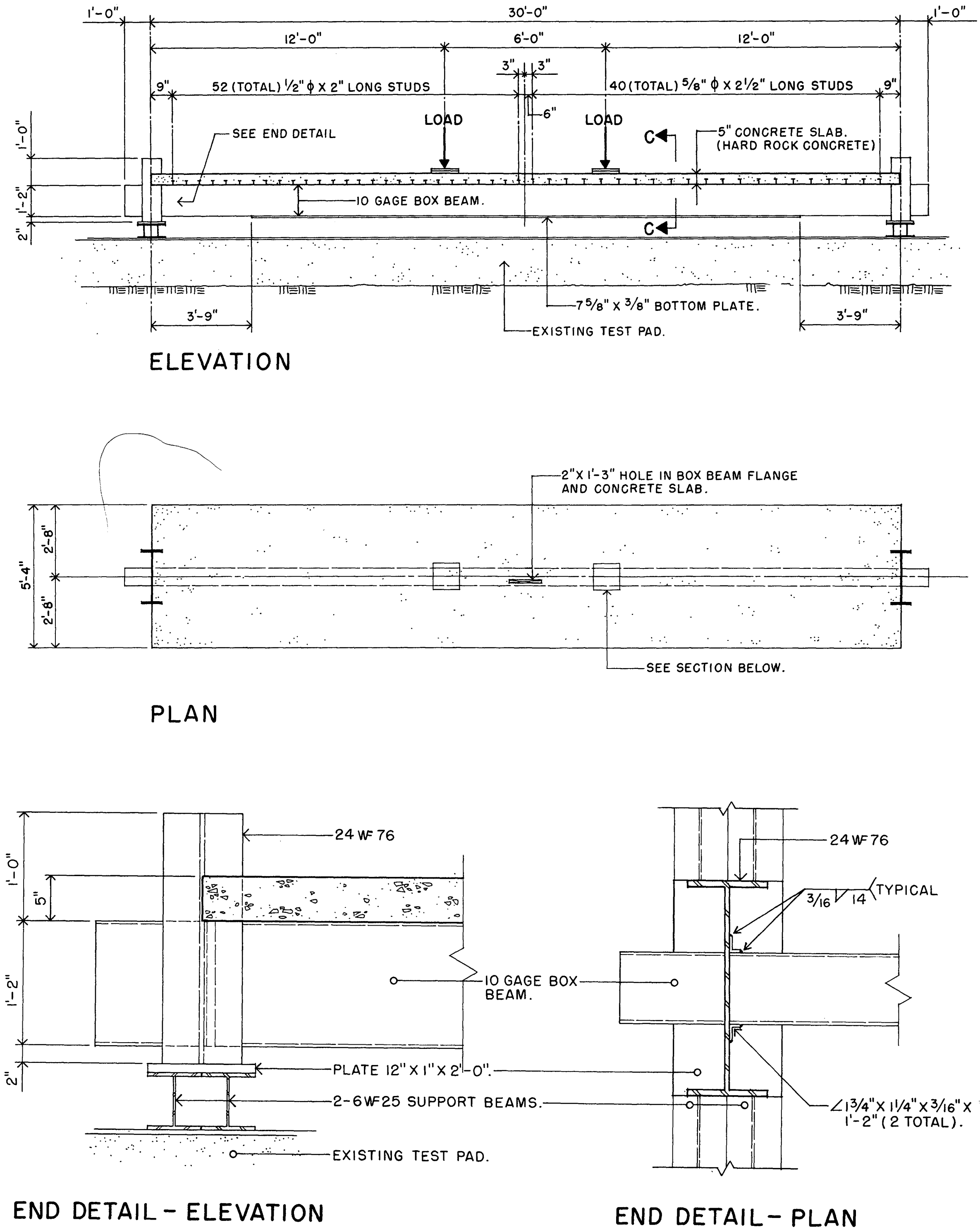

Fig. 6. Light gage duct-beam details

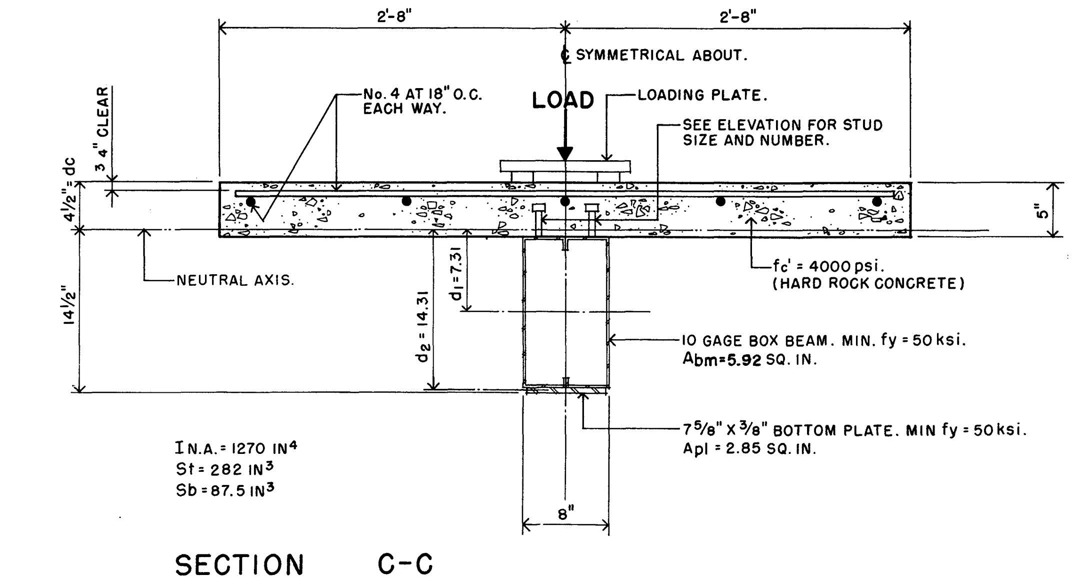

Fig. 7. Test beam Section C-C

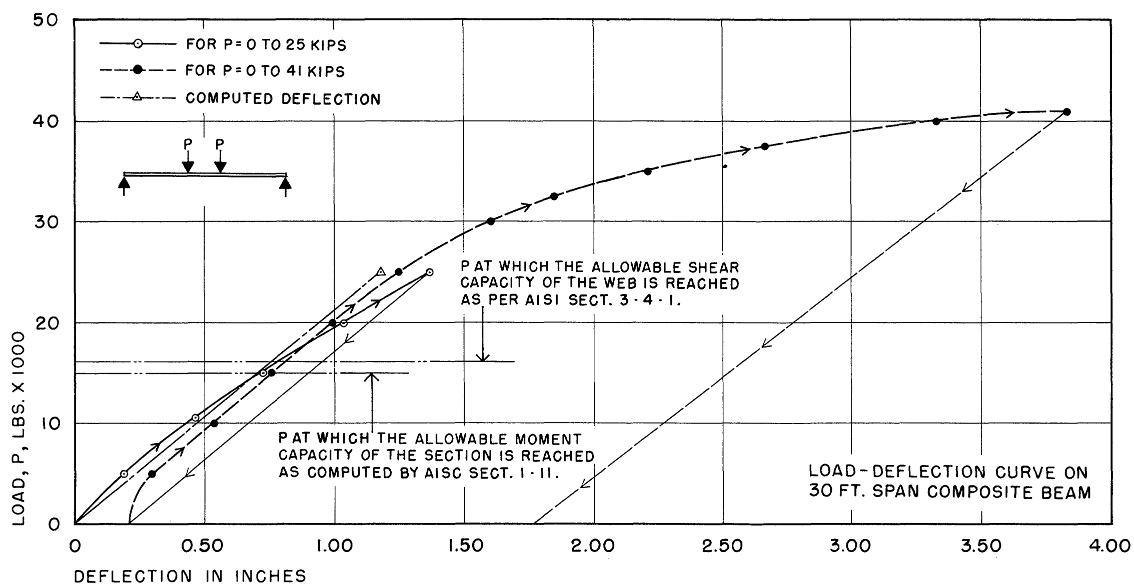

Fig. 8. Light gage composite duct-beam test results

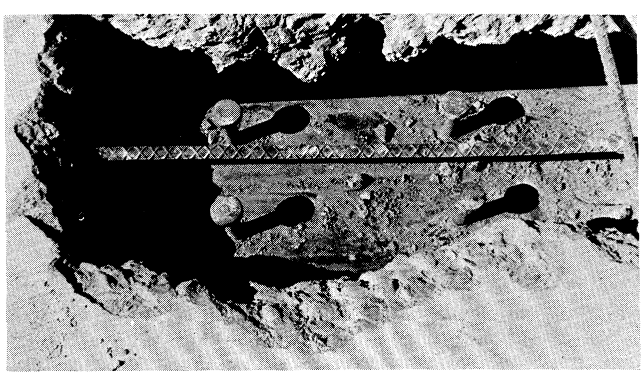

Fig. 9. Condition of studs after test

the shoring was removed from the underside of the beam, whereupon the beam deflected $\frac{3}{16}$-in. The test load was applied through two calibrated hydraulic jacks. When the test loads approached 25 kips at each point of application, they were released and the residual deflection was measured (Fig. 8).

Upon reloading, an increase in stiffness was observed, presumably due to the redistribution of contact pressures between the studs and the concrete.

Theoretically, the yield point in the bottom flange should be reached at $P = 26$ kips. The test result corroborates this, as the deflection curve starts flattening out at approximately 25 kips.

Ultimate load was reached at $P = 41$ kips, which corresponds to a maximum moment in the beam of

$$(P \times X) + \frac{WL}{8} = (41 \times 12) + \left(\frac{10.8 \times 30}{8}\right)$$
$$= 492 + 40.5 = 532.5 \text{ kip-ft}$$

where W = dead weight of test beam.

For comparison, the theoretical ultimate capacity is computed as follows (see Fig. 8):

$$M_{bm} = A_{bm} \times f_y \times d_1 = 5.92 \times 50 \times 7.31$$
$$= 2160 \text{ kip-in.}$$

$$M_{pl} = A_{pl} \times f_y \times d_2 = 2.85 \times 50 \times 14.31$$
$$= 2040 \text{ kip-in.}$$

$$M_{\text{steel}} = M_{bm} + M_{pl} = 2160 + 2040 = 4200 \text{ kip-in.}$$

$$A_s = A_{bm} + A_{pl} = 5.92 + 2.85 = 8.77 \text{ sq in.}$$

$$F_s = A_s f_y = 8.77 \times 50 = 438.5 \text{ kips} = F_c$$

$$M_{\text{conc}} = F_c \times \frac{d_c}{2} = 438.5 \times 4.5/2 = 985 \text{ kip-in.}$$

$$M_{\text{ult}} = M_{\text{steel}} + M_{\text{conc}} = 4200 + 985 = 5185 \text{ kip-in.}$$

or

$$M_{\text{ult}} = 432 \text{ kip-ft} = M_{\text{D.L.}} + M \text{ due to } P_{\text{ult}}$$

$$P_{\text{ult}} = \frac{M_{\text{ult}} - M_{\text{D.L.}}}{12} = \frac{432 - 40.5}{12} = \frac{391.5}{12}$$
$$= 32.6 \text{ kips}$$

It is seen that the actual ultimate load exceeds the theoretical by about 25 percent, probably due to the effect of strain-hardening and higher strength of concrete than specified.

The theoretical safety factor (ultimate vs. working load) is $432/219 = 1.97$. The actual safety factor is $532.5/219 = 2.43$. No distress or buckling was noted in the beam webs, which at ultimate load had an average stress of $(41 + 5.4)/2 \times 14 \times 0.1345 = 12.2$ ksi in shear and were yielding in tension as evidenced by visible yield lines (Lueder's effect) on both webs.

Upon removal of the load, a residual deflection of about 1.77 in. was observed.

After the test, the concrete in the top flange was jackhammered out to expose the studs in a region of high shear. No visible deformation of the stud or beam flange was noted (Fig. 9). The maximum force in the concrete, based on the observed ultimate moment, is computed to be 542 kips. Dividing this by the number of shear connectors on each half of the beam,

$$542/52 = 10.4 \text{ kips}/\tfrac{1}{2}\text{-in. stud}$$

and

$$542/40 = 13.5 \text{ kips}/\tfrac{5}{8}\text{-in. stud}$$

This compares very closely with the ultimate values obtained from the pushout tests and discussed previously. A point of difference is that there was no visible deformation of either the concrete or the stud in the full scale test, while considerable deformation of the stud occurred in the pushout tests. There were no signs of cracking or any distress in the region of the hole.

Conclusions—The performance of this high-strength light gage composite beam followed standard composite theory very well. The web strength proved more than adequate, and the deflection characteristics are also quite acceptable. (At full dead + live load of 50 psf, deflection = 0.75 in. = $L/480$.)

The use of light gage composite beams appears quite feasible, therefore, and may be considered as an alternative to W composite design. Of course, for long spans (on the order of 40 ft and above) the web shear starts governing, and it becomes necessary to either reinforce the web or use thicker plate material.

It is impossible to draw a line above which one has to use structural plate, as each case will have its own special conditions that will have to be met. It seems, however, that for some load-span ratios, the light gage composite duct-beam may offer an economy in construction.

EFFECTS OF WEB HOLES

Shear Strength of Beams—In some types of application of the duct-beam system, it may be desirable to connect beams in such a way that air can be moved from one beam to another. Also it may be necessary to exhaust or supply air through the side of the beam instead of from the top. In either case, holes in the webs would be required. Since there is little information in engineering literature on this subject, it was decided to make some tests. Several types of holes were cut in 8 x 14-in. duct-beam specimens, the details and test results of which are illustrated in Figs. 10 through 16.

Specimen No. 4, a 10 ga. beam with a 9-in. dia. circular hole in each web, reached a load of 38 kips before it failed (see Figs. 10 and 11, load point 2). The shear on the cross section at the hole was 26 kips, giving an average stress of 22.7 ksi at the hole. This is very near the yield point of the steel in shear (29 ksi). When the load point for this specimen was near the unreinforced circular hole, the web around the hole started buckling and, when the load point was away from the circular hole, the beam web at the column support nearest the load point showed warping. The ultimate load in the latter condition was 70 kips, with shear on the web cross-section at the support nearest the load point of 48 kips, giving an average stress of 13.4 ksi. (Note the allowable shear stress as per AISI is 5.9 ksi for $h/t = 104$; which gives a safety factor of $13.4/5.9 = 2.28$.)

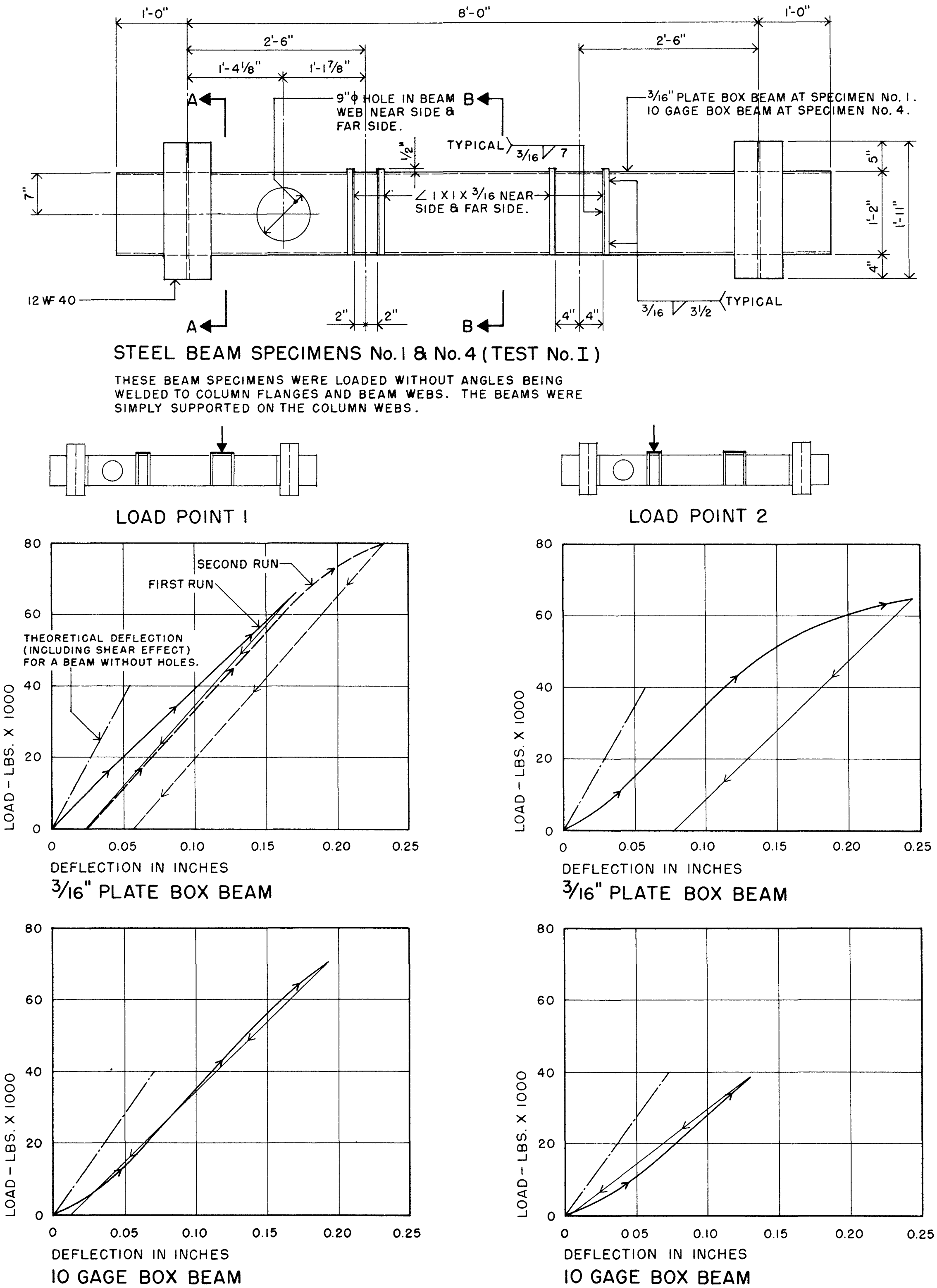

Fig. 10. Beam details with round holes

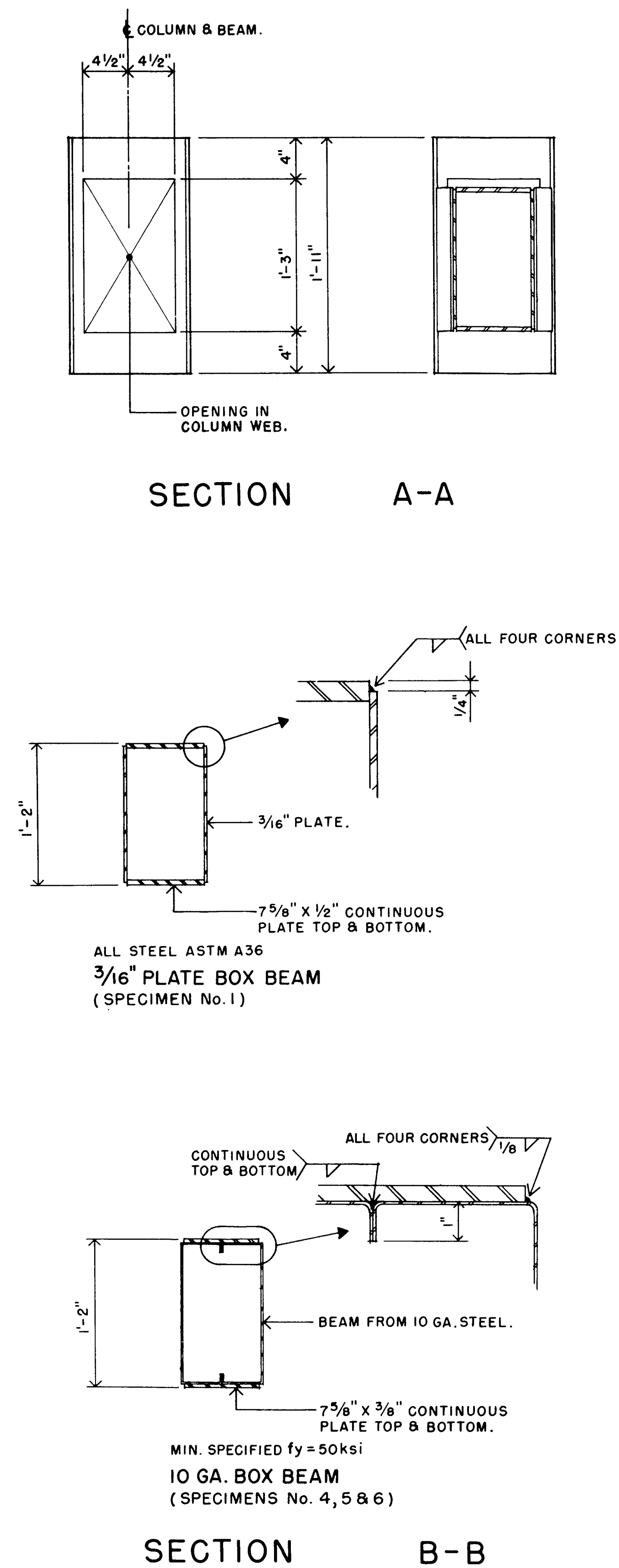

Fig. 11. Typical beam details, Sections A-A and B-B

Specimen No. 5, a 10 ga. beam with an oval shaped hole in each web, reached a load of 60 kips before it failed. The 8-in. dia. hole was reinforced and the oval hole was stiffened with angles which were welded to simulate a beam-to-beam connection as detailed in Fig. 12. The ultimate shear on the cross section at the oval hole was 36.2 kips, giving an average stress of 33.6 ksi. The failure was by buckling of the web at the hole.

The test of Specimen No. 6, a 10 ga. beam with unreinforced 9-in. dia. holes and reinforced oval holes in each web, gave results not appreciably different from those of Specimens No. 4 and 5. The data is shown in Fig. 13. The failure mode of the webs is shown in Fig. 14.

Of unusual interest is Specimen No. 4 (Figs. 15 and 16), which incorporates a hole 12½ x 9 in. in a beam of 14-in. depth. This would permit the full cross section to be utilized for air handling. The performance of the "full size" hole with a C-shaped collar showed a high shear capacity. There was no evidence of buckling of the collar, even after yielding of the collar started. Also, the web at the support nearest to the load point started buckling. The ultimate shear across the collar was 64 kips, i.e., the collar had an average shear stress of 12.8 ksi, when it yielded at the inner corners.*

For purposes of comparison of the actual and the theoretical deflections (including bending and shear), both curves are shown on the load deflection diagrams. Obviously the effect of a hole is to reduce somewhat the rigidity of a beam. As an example of the effect of the holes on a beam's stiffness, we can approximate the effect as follows: Assume a 30-ft long composite beam with a 10-ft wide tributary area. The bending deflection of the beam is approximately 1 in. at full dead + live loads (150 psf total). Assuming 9-in. dia. unreinforced holes in both webs at quarter points, the shear at the holes is 11.25 kips. From Fig. 10, for a 10 ga. beam loaded at point 2, the difference in deflection due to the hole is approximately 0.033 in. For holes at each quarter point, the total increase is 0.066 in. The total deflection is obviously very little affected (by about 7 percent). The test results for the 3/16-in. specimens generally showed similar performance. The thicker plate produced higher ultimate loads even though the yield point was 36 ksi vs. 50 ksi for the light gage. Fig. 10 shows the test results.

Column Test—In order to allow for the passage of duct-beams through columns, it is necessary to cut a hole in the web of the column. While it is possible to calculate the allowable stresses and loads on the remaining cross section, it was decided to verify the results with a test.

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* *A possible extension of this idea to the problem of reinforcing a large hole in a WF would be as follows. A plate would be welded to the top flange in the line of web and extended past the hole to create a "Vierendeel" effect.*

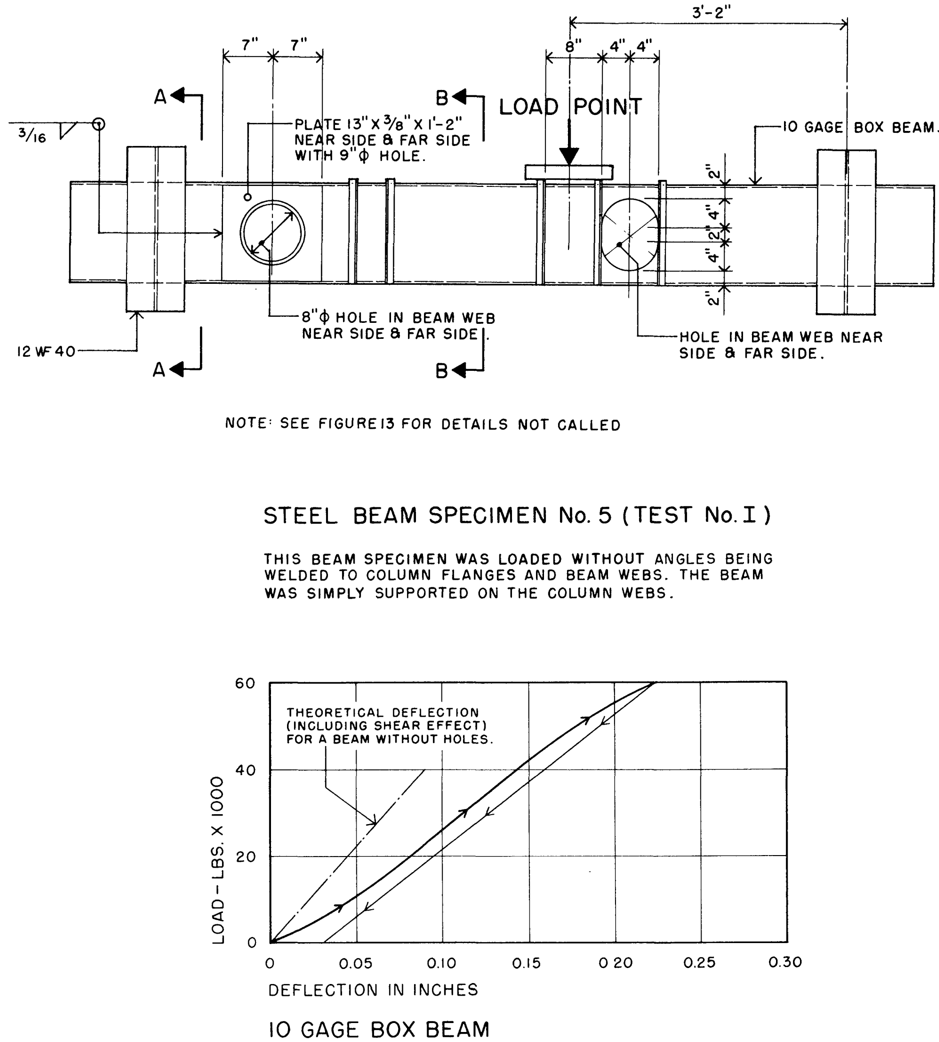

Fig. 12. Beam details with reinforced round holes and oval holes

Figure 17 shows a sample test specimen cut from a 12WF40 after the compression test. The size of the hole was 9 x 15 in. The L/r of the remaining stem section (based on L = 15 in.) was 75. Therefore, for A36 steel, P_{all} = 15.90 × 8.8 = 139 kips. The computed ultimate strength was 246 kips.

The first specimen failed at a load of 267 kips. The other two were loaded up to the limit of the testing machine (295 kips) without any visible deformation. The safety factor is 267/139 = 1.93.

The allowable load on the column without the web cutout is computed to be (for L = 12 ft) 188 kips. The efficiency of the column with the hole is, therefore, (139/188) × 100 = 74 percent as compared with the full cross section.

APPLICATION OF DUCT-BEAM SYSTEMS

In the short time since the duct-beam concept was developed, it has been possible to apply the system to several projects in the design stage.

Figure 18 shows a portion of a school building that was originally designed by our office as a WF composite structure, with a bay size of 28 ft x 28 ft. Figure 19 shows the conventional supply duct network and Fig. 20 shows the ductwork required to complete a working system, if the beams and girders are duct-beams, maintaining the same ceiling diffuser locations.

Figure 21 shows a multistory high school* with a bay size of 32 x 64 ft. The larger than usual spans permit

* *Diamond Heights High School, San Francisco, Calif., Reid & Tarics, Architects & Engineers.*

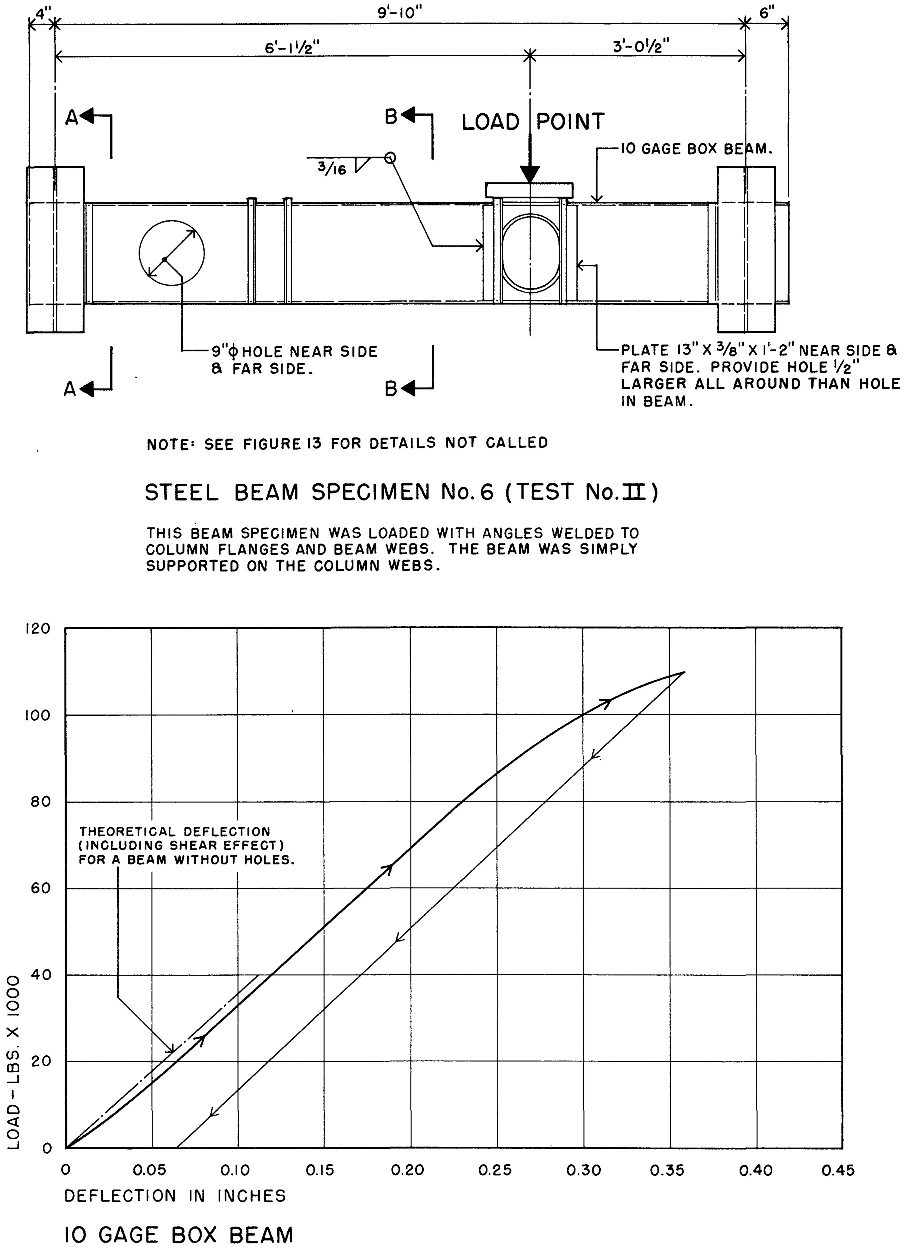

Fig. 13. Beam details with round holes and reinforced oval holes

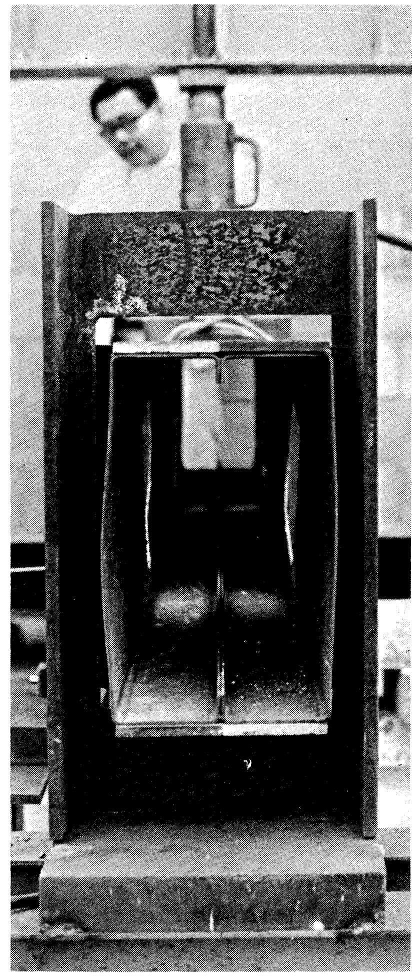

Fig. 14. Buckling of beam webs

an increased amount of flexibility for future changes in room layouts. The movable partitions will be an integral part of the air supply system, taking the air up from the beam and discharging through a diffuser into the room at the required height. In large spaces, a flexible duct connected to the top of a partition and running above the ceiling will provide efficient air distribution.

Figure 22 shows a typical plan of a four-story nursing home project.* The structure is girdled by horizontal ducts, one side of which is the spandrel beam. The 55-ft long clear span duct-beams tap into the exterior duct through the spandrel beam web and distribute the air into the interior. The return air is similarly handled from the other side of the building. It is expected that the partition system will perform air handling in a manner similar to the method described above. Note that the clear span again permits the maximum architectural flexibility for interior layout and possible future changes in the building.

Figure 23 shows a dormitory or apartment house structure. The bathroom chases are used as vertical duct shafts. Air is distributed through the duct-beams and introduced up into the rooms at the exterior win-

* *National Medical Association, Foundation Nursing Home.*
Architects: Building Systems Development, Inc., Rex Whittaker Allen & Associates, R. J. Nash, A.I.A.
Consultants: Reid & Tarics

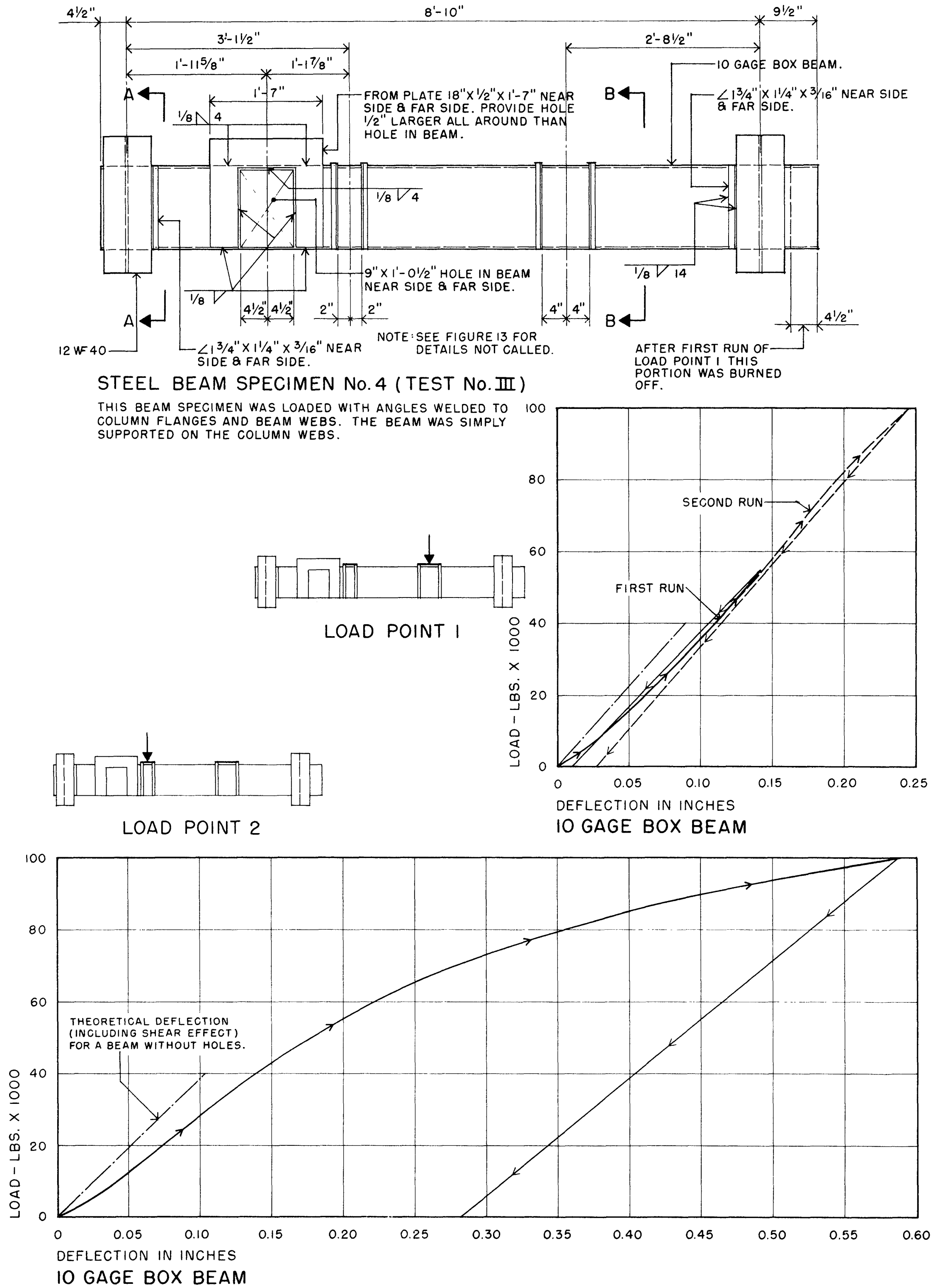

Fig. 15. Beam details with "full size" hole

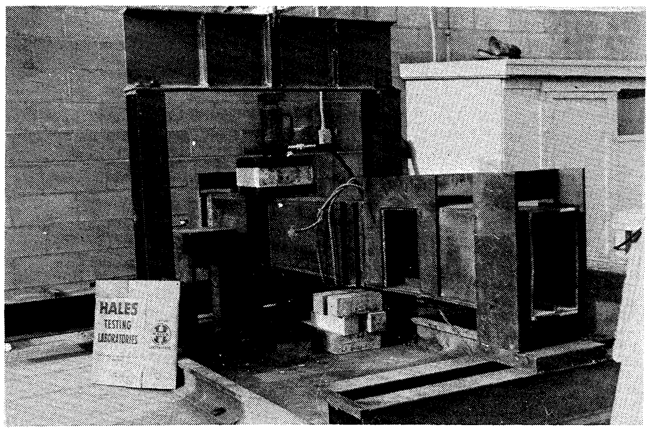

Fig. 16. Specimen with "full size" hole during test

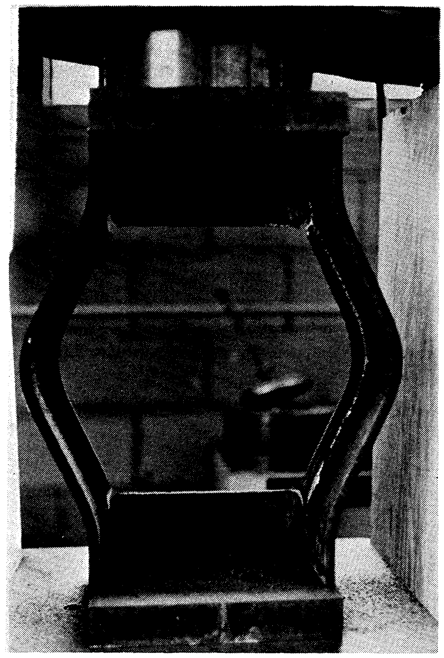

Fig. 17. Column with web cutout after test

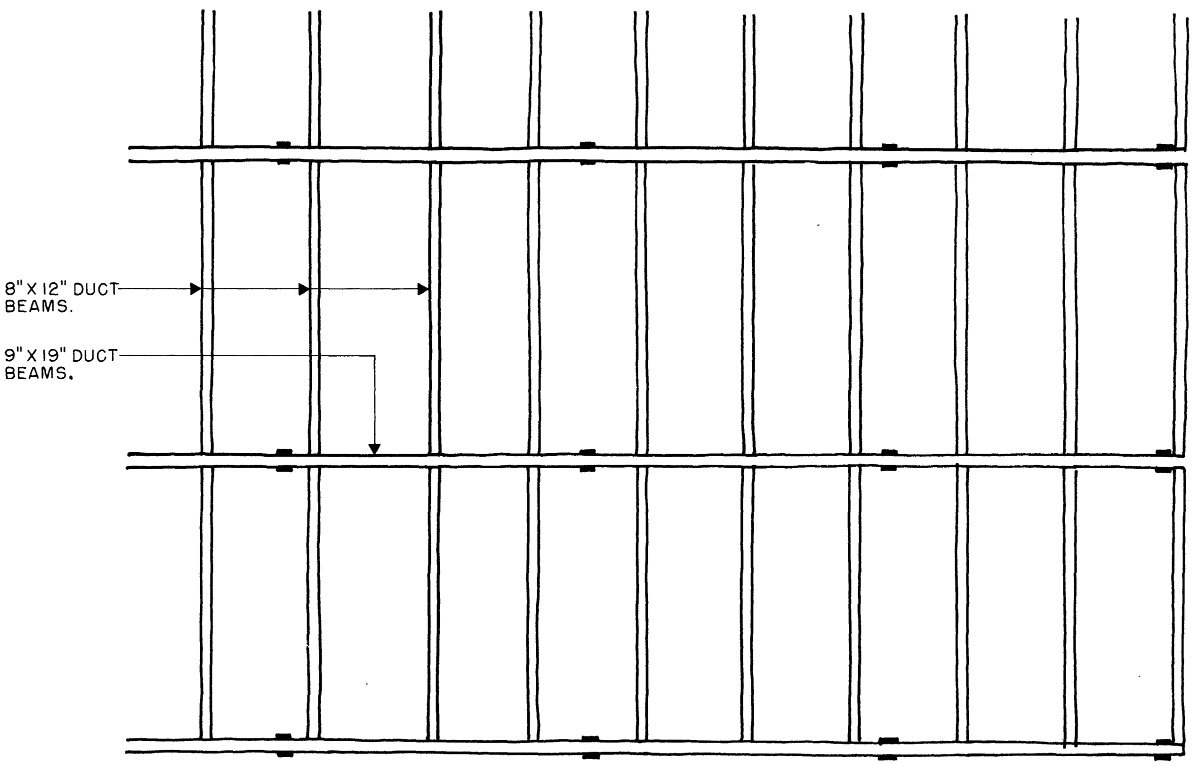

Fig. 18. Duct-beam framing layout

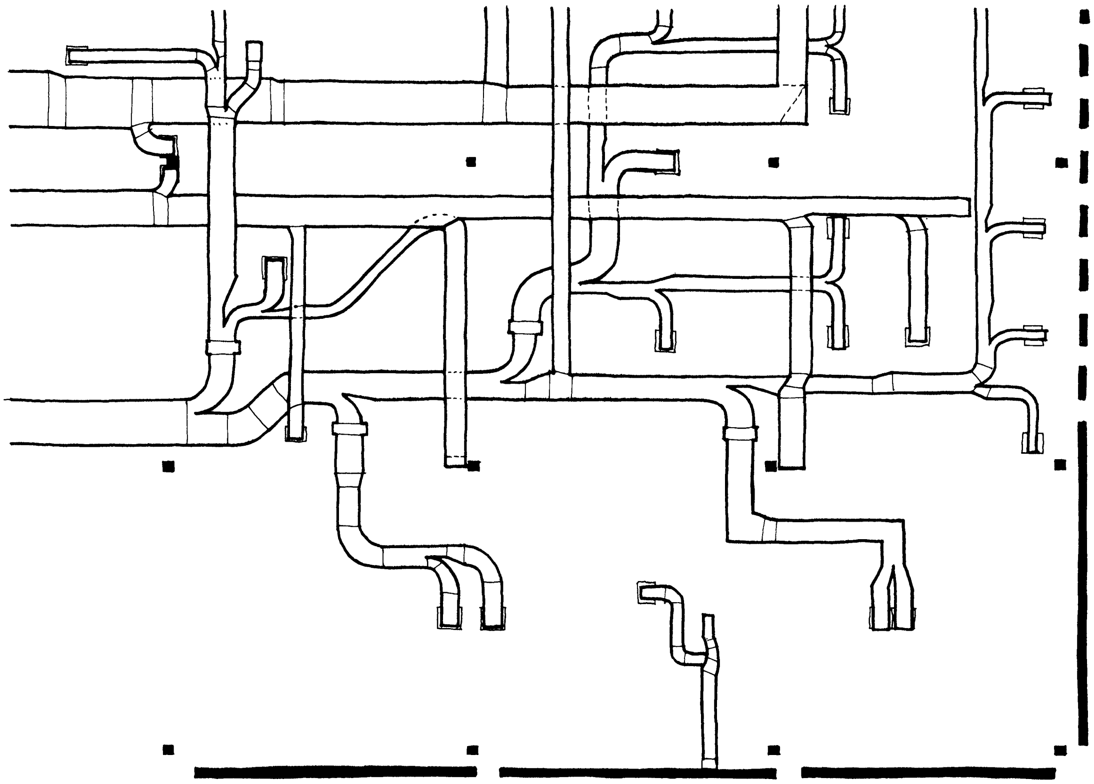

Fig. 19. Conventional supply ductwork

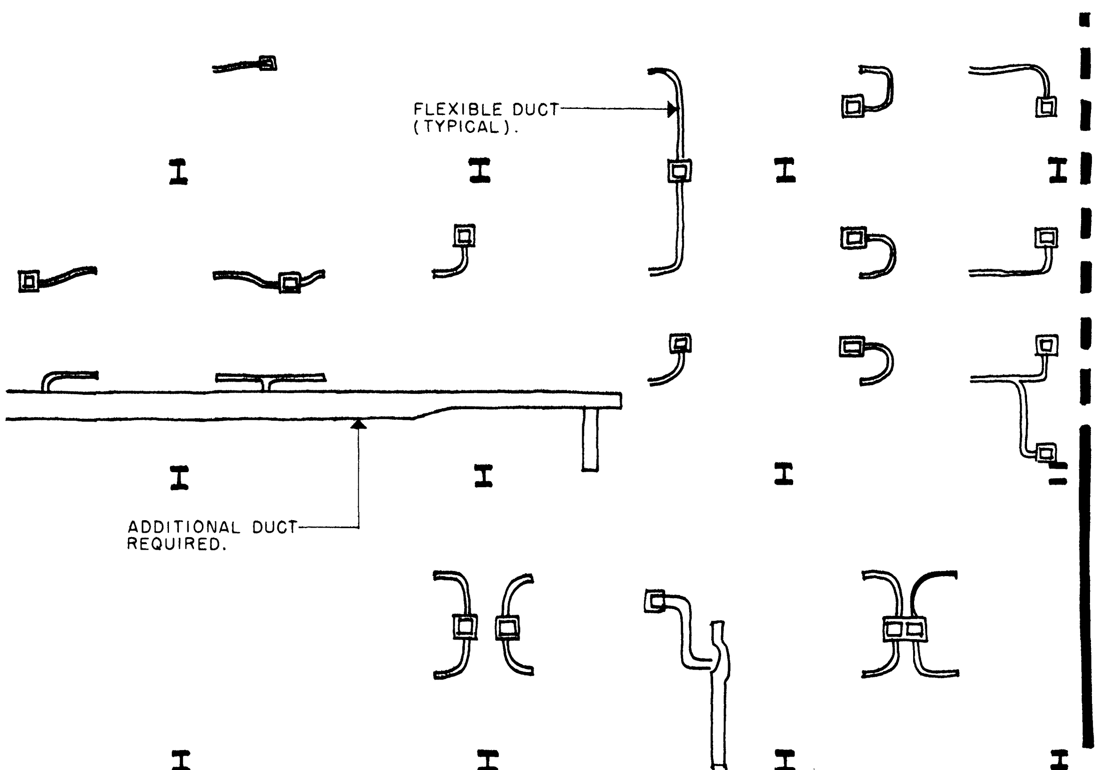

Fig. 20. Additional ductwork required with duct-beam system

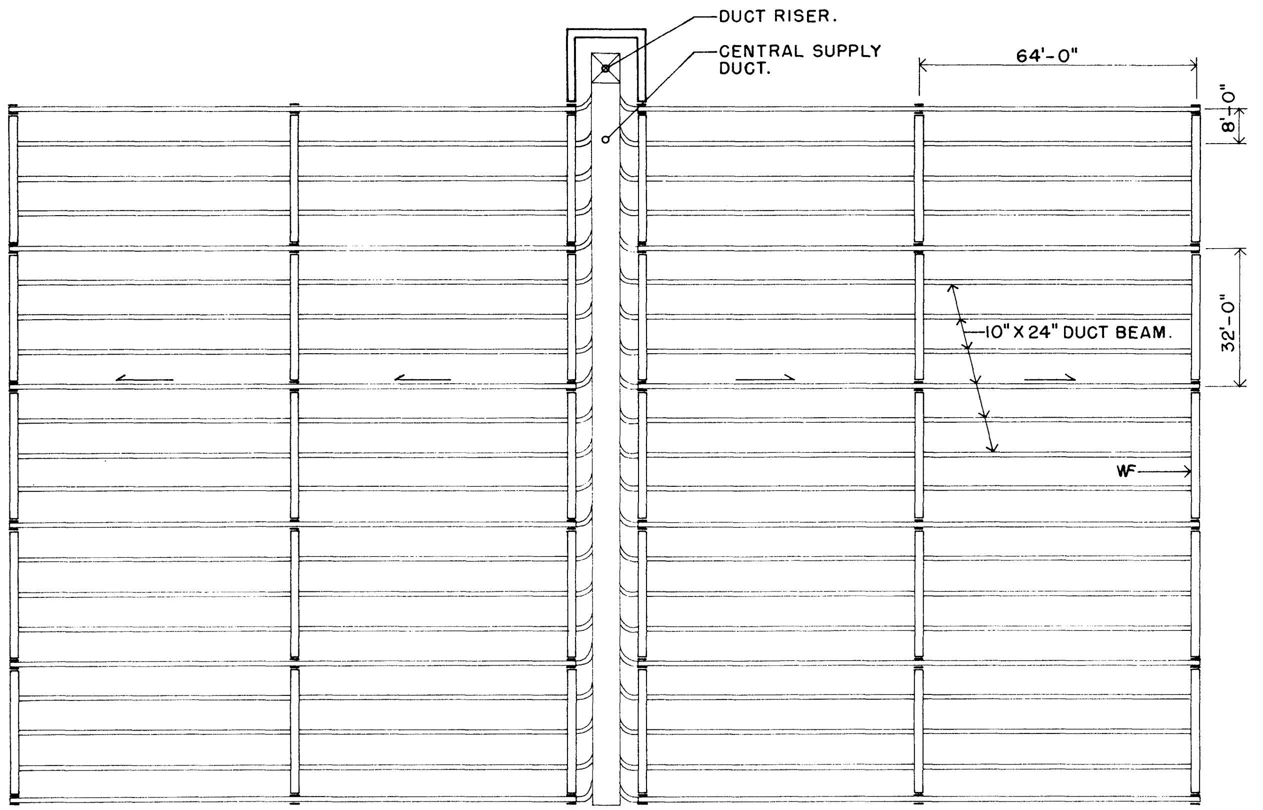

Fig. 21. Diamond Heights High School duct-beam framing plan

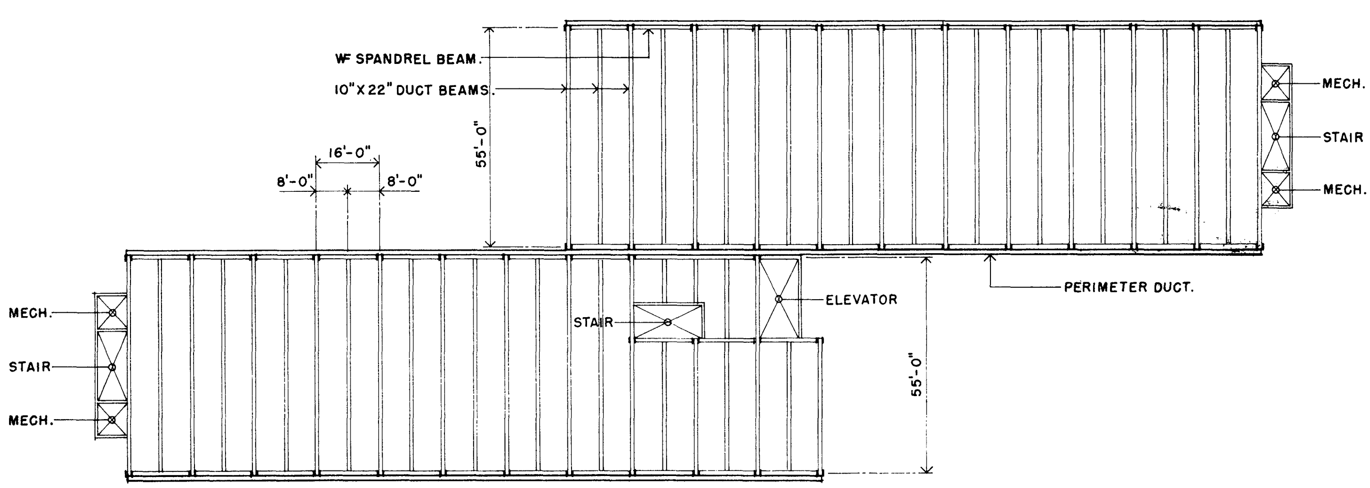

Fig. 22. Nursing home duct-beam layout

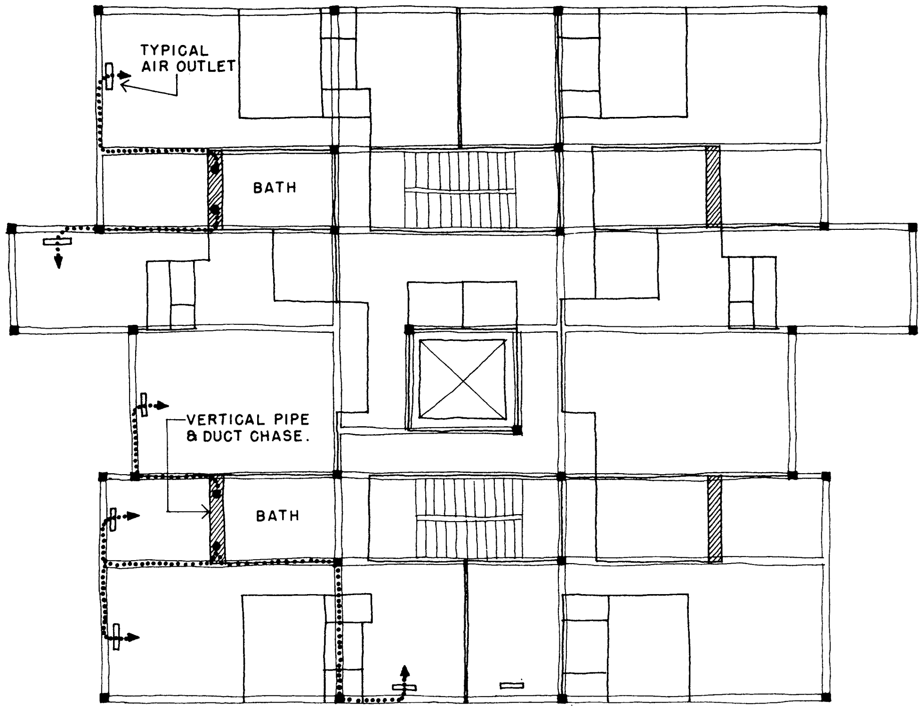

Fig. 23. Apartment house or dormitory duct-beam layout

dows. The return is through a ceiling plenum or, if the ceiling is omitted, through louvered doors into corridors and back to the bathroom duct chases.

A detail not previously mentioned is a column detail that allows a two-way beam intersection with full air interchange in both directions. This permits two-way framing such as shown on Fig. 23 to be used without impairing air circulation.

SUMMARY

While it is too early to assess the overall economy of the duct-beam systems, some features of the concept are worth mentioning. Like any composite design system, it lends itself easily to long spans. As the span increases, so does the depth, and thereby the inherent air capacity of the system, shifting the point of optimum economy towards the architecturally more desirable longer spans. The overall story height of a building may be reduced in some applications and in the case of apartment buildings with stacked bathrooms it is possible to omit suspended ceilings in the living areas.

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- John Schwafel, AISI
- Fred Comee, United States Steel

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