LOAD DISTRIBUTION TEST ON PRECAST HOLLOW CORE SLABS WITH OPENINGS

Ted Johnson
Houdaille Duval Wright Co.
A Division of Houdaille Industries, Inc.
Jacksonville, Florida

Zohair Ghadiali
Houdaille Duval Wright Co.
A Division of Houdaille Industries, Inc.
Jacksonville, Florida

This test investigates the distribution of loads on precast hollow core slabs in areas where openings exist. The test panel is made up of four precast units, connected by grouted shear keys, with a 40 x 40-in. opening centered in the panel. The results verify that full load distribution is attained up to the ultimate flexural capacity of the system.

In many applications of prestressed concrete hollow-core slabs as floor systems, it is necessary to provide large openings through some units for mechanical and plumbing requirements. Openings through an individual unit may require removal of as much as 50 percent of the cross section. In most cases, this design can be accomplished by transferring the load to adjacent full units.(1),(2) This investigation covers a load test which was conducted to assist in verifying such a design approach. A floor system of 6 in. thick hollow-core units with a 40 x 40-in. cutout was used for the test (see Fig. 1).

In the investigation the following points were of interest:

1. What is the capacity of the system?
2. How effective is the shear key in transferring load?
3. What type of local cracks result before failure?
4. What type of failure results at ultimate load?

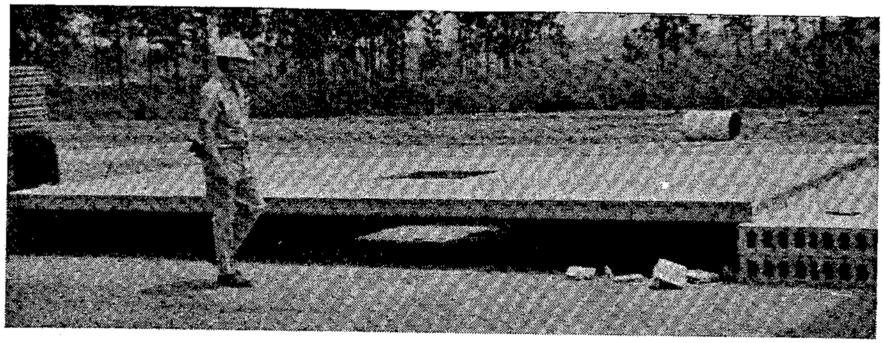

Fig. 1. Test assembly prior to loading

5. What is the most critical section?

Materials and test setup

The hollow-core units selected for the test had the following properties (see Fig. 2).

1. Slabs 6 in. thick x 40 in. wide x 23 ft long.
2. Lightweight concrete: 120 lb per cu ft with a 28-day strength of 5000 psi.
3. Prestressing strand: ten 5/16 in. diameter; 250-kip strands with an initial tension of 10,150 lb each; $F_e = 76.1$ kips; and $e = 1.350$ in.
4. Weight of units = 40 psf.
5. Age of units at the time of testing = 35 days.

The framing plan was 23 ft x 13 ft 4 in. as shown in Fig. 3 with the supports being two 8 in. thick hollow-core units. No connection was made at the supports.

A 4000-psi sand and cement mix was used to grout the keyways 8 days prior to testing. The strength of the grout at the time of testing was 3660 psi.

The loading elements were 8 x 8 x 16 in. concrete block weighing 39 psf and 6 in. hollow-core slabs, 3 ft 4 in. x 5 ft 7 in., weighing 740 lb each.

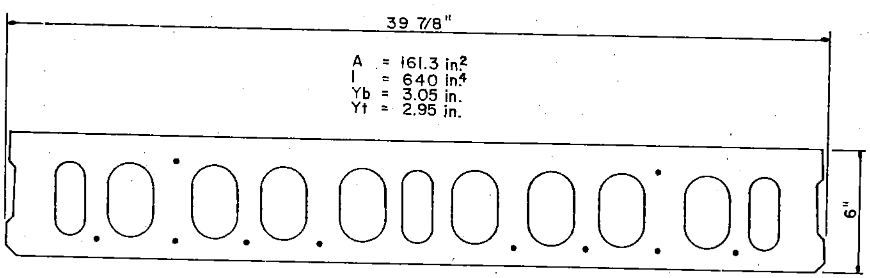

Fig. 2. Typical span section

Load considerations

The design span was 22.75 ft at which a full width unit has a calculated working capacity of 74 psf in addition to its dead load of 40 psf. The calculated ultimate capacity of the unit using ACI 318-71[3] is 179 psf. Assuming uniform transfer of load from the units with cutouts to the adjacent full-width units, the test system has a resulting section of:

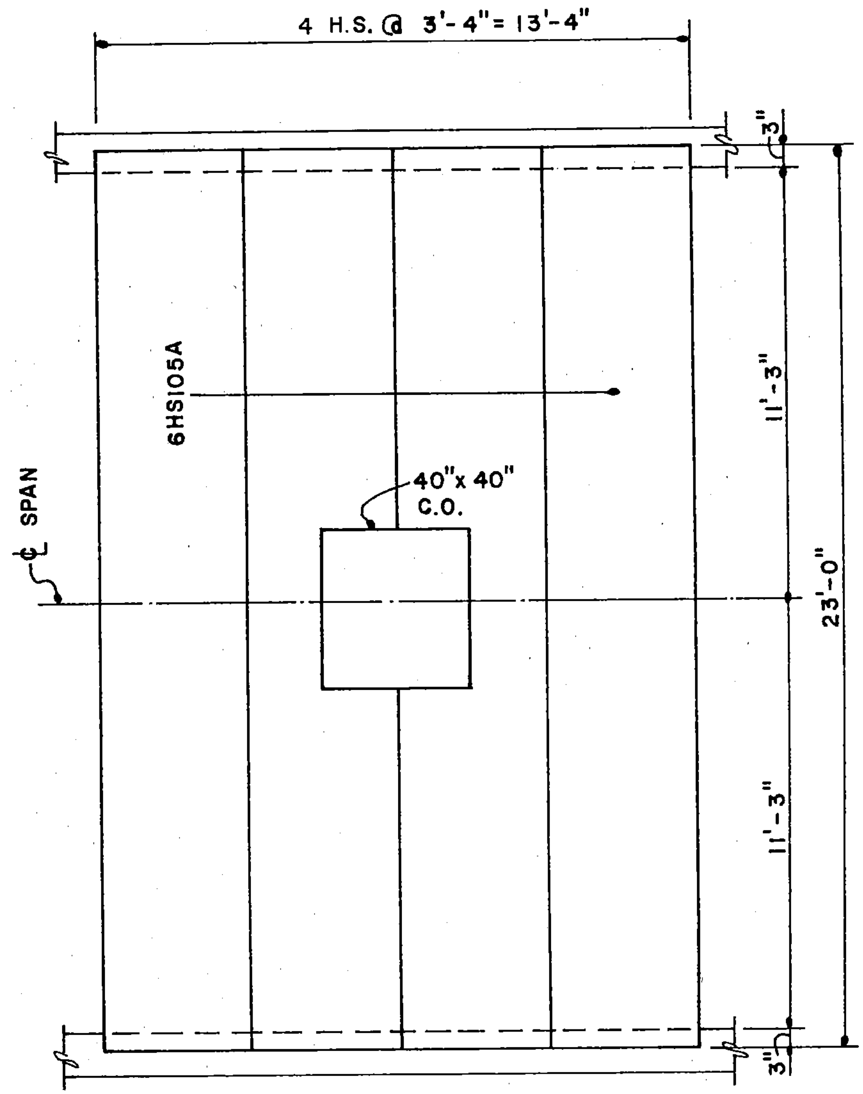

Fig. 3. Layout of hollow-core test assembly

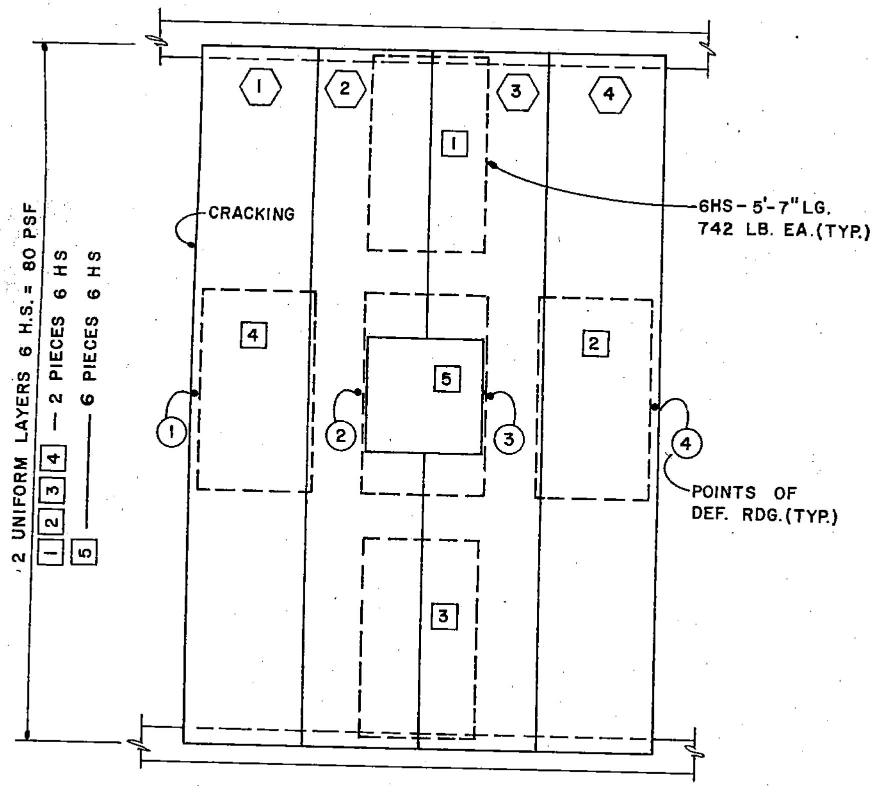

Fig. 4. Loading pattern for ultimate load test

$$\frac{160-40}{160} = 75 \text{ percent}$$

of a full section.

Thus, the working capacity of the system is:

$$0.75\,(40+74) = 85.5 \text{ psf}$$

This capacity is made up of 40 psf dead load and 45.5 psf live load. Based on Section 20.4.3 of the ACI 318-71 Building Code, the system must successfully perform under a total test load of:

$$0.85\,(1.4 \times 40 + 1.7 \times 45.5) = 113 \text{ psf}$$

Therefore, the superimposed test load is 113 – 40 = 73 psf.

Test procedure

Initial deflection readings were taken at four points at midspan as shown in Fig. 4. The readings were taken with an engineer's level and rod. Next, the units were uniformly loaded with two layers of concrete block and an additional 21 blocks were placed at each end for an equivalent superimposed load of 84 psf (Fig. 5). This load was greater than the required test load of 73 psf and was 185 percent of the design load of 45.5

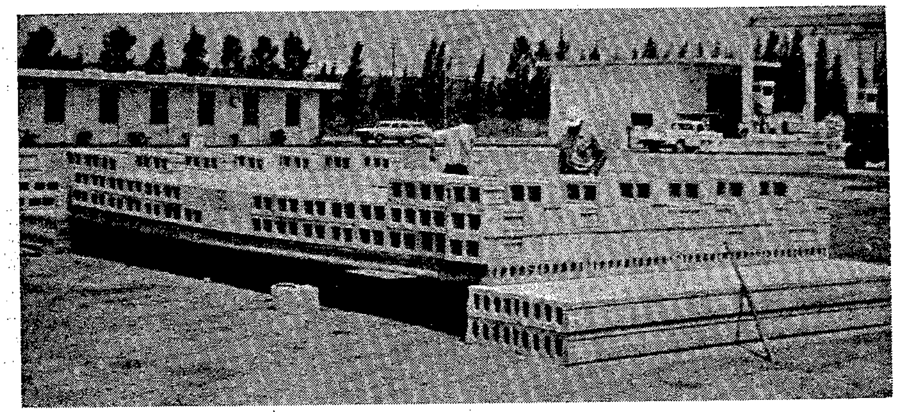

Fig. 5. Assembly under full test load

psf.* Deflection readings were taken at this time and, as shown in Table 1, no differential deflection occurred between the units.

Inspection of the units revealed three vertical flexural cracks in Unit 1 at about midspan. One large crack extended 5 in. up from the bottom, the others about 2 in. The large crack also extended across Unit 2 and terminated at the cutout. Unit 4 had one small crack (about 2½ in. high) at midspan.

The different performances between Units 1 and 4 could probably be attributed to strand slippage in Unit 1. The three outside strands in Unit 1 had approximately ¼ in. of slip at one end. This condition had been noted prior to the application of the test load.

The test load was allowed to remain on the units for 24 hr and was then removed in two steps. First, the load on the two outside units was removed to investigate the effects of differential loading (Fig. 6). Under this condition, the four units continued to work together, exhibiting uniform deflection (see Table 1). Visual inspection of the grout keys revealed no distress in the precast units or the grout keys. The remainder of the load was then removed

**The test was originally setup to follow the ACI 318-63 Code provisions. The paper was subsequently changed to reflect the 1971 Code.*

Table 1. Midspan deflection, in.

Point	Dead load reading	Test load (84 psf)	One-half test load (42 psf)	Test load removed
1	0	1⅞	1	0
2	0	1 15/16	1⅛	⅛
3	0	1⅞	1	0
4	0	1⅞	1	0

Fig. 6. Reading deflections at differential loading stage

and deflections were noted. The large crack in Unit 1 was still slightly visible, but not open, on removal of loads. Since recovery of the units was complete, the requirements of Chapter 20 of ACI 318-71 had been met.

Ultimate capacity test

To determine the ultimate capacity of the units, they were loaded with 5 ft 7 in. x 3 ft 4 in. pieces of 6 in. hollow-core slabs (Fig. 7). The total load at

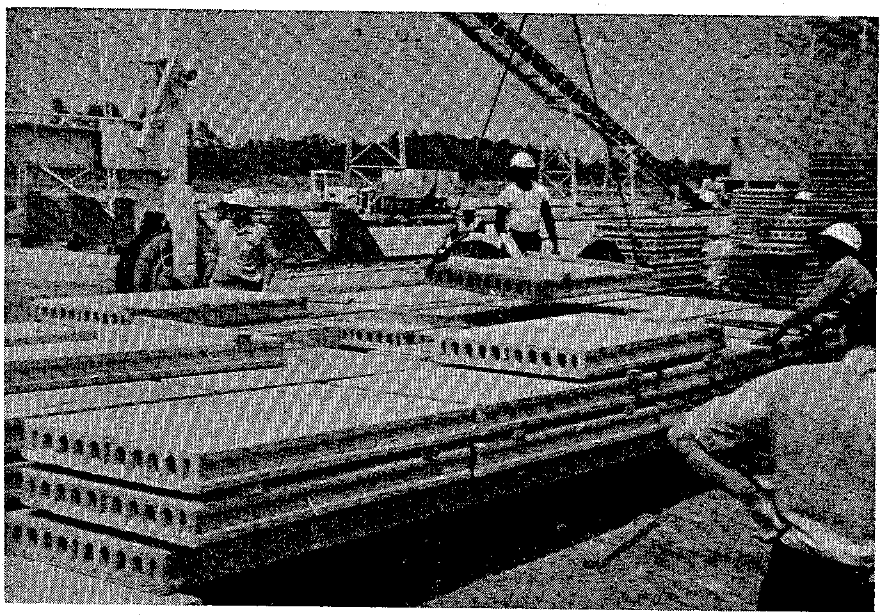

Fig. 7. Assembly being loaded to determine ultimate capacity

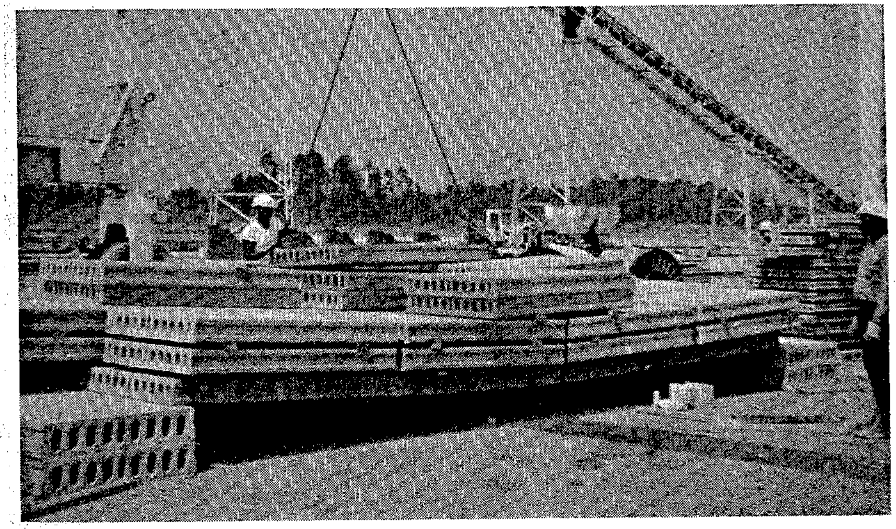

Fig. 8. Assembly at final loading stage of ultimate capacity test

failure and the load sequence was as follows (see Fig. 4):

1. Two uniform layers of 6 in. hollow-core over the full area.
2. Two units in Positions 1, 2, 3, and 4, respectively.
3. Six units in Position 5, one at a time.

Failure occurred when Unit 6 was placed in Position 5 (Fig. 9). The maximum sustained deflection prior to failure was 9 in. (Fig. 8). The units failed in flexure, with no separation occurring through the shear keys prior to failure.

Discussion of test results

Of primary consideration in this test was the ability of the system to provide lateral distribution of loads from the units with reduced sections to the units with full sections. The ability of the system to achieve this was investigated by taking deflection readings along the midspan line at various loading stages. The deflection data plus visual inspection of the grout keys provided evidence of the successful lateral distribution of the loads. The system's successful performance was particularly apparent during the differential loading stage when all superimposed loads were removed from the units with full sections while the full test load remained on the units with reduced sections. (This load was 138 percent of the ultimate capacity of these units acting alone.)

To provide a sound basis for future design, both the ACI test procedure and the load to destruction were used in investigating the system. The system's performance was quite good in both cases. Under the ACI test, all requirements for load capacity and recovery of deflection were met. In the ultimate load test, the load at failure was 20 percent greater than the calculated

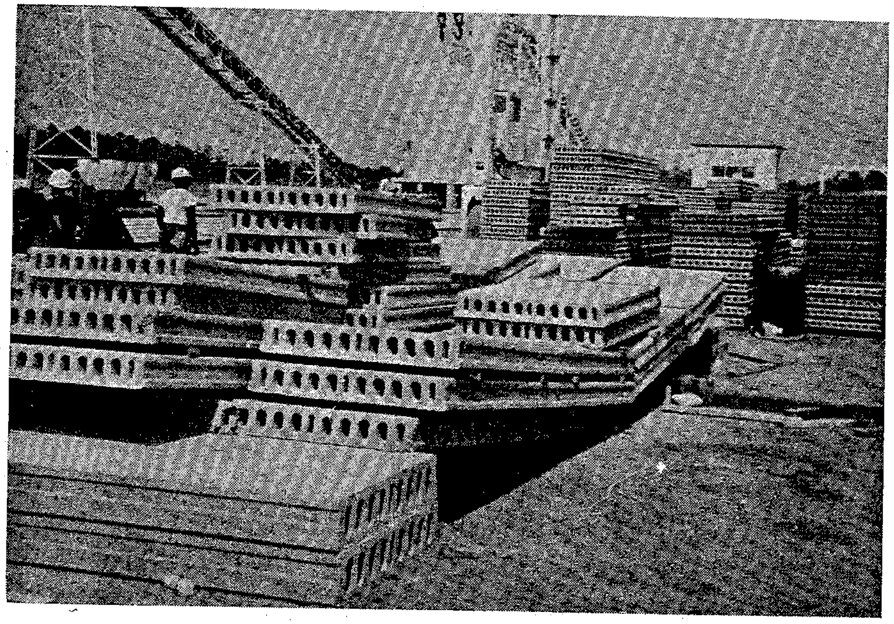

Fig. 9. Failure of system

design ultimate load per ACI 318-71 (see Appendix).

Conclusions

The principal findings of the load test were as follows:

1. The shear keys were effective in providing load transfer across the full panel width up to ultimate flexural failure. Concrete topping or mechanical weld plates are apparently not required for this action.

2. The successful performance of the shear keys took place without the presence of end anchorages to resist lateral forces. Therefore, special slab anchorages or border restraints are not required in the design of panel systems such as the one tested.

3. The presence of the 40 x 40-in. cutout without special edge reinforcement did not lead to premature failure of the system. In such a system, openings up to a full panel width can be provided without special reinforced curbs or headers.

4. The system provides adequate safety factors as shown by the following observations:

(a) The ultimate load requirement was met.

(b) Considerable additional load was supported after first cracking.

(c) Large deflection occurred as a warning that the ultimate capacity was being approached.

5. The test results further confirm previous findings on load distribution in hollow-core construction.[1] It may be concluded with reasonable assurance that the design method presented in this paper (see Appendix) for openings in hollow-core units is valid.

Final remarks

This study is based on a field test of hollow-core prestressed concrete slabs. The field test was conducted in a prestressed concrete plant using materials and equipment that are normally available at this type of facility. This test procedure cannot compare with the rigidly controlled techniques of laboratory testing. It is felt, however, that field testing, when coupled with recognized methods of analysis, can provide useful information to the designer.

References

1. ACI Committee 711, "Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)," American Concrete Institute, Detroit, 1958.
2. LaGue, David J., "Load Distribution Test on Precast Prestressed Hollow-Core Slab Construction," *PCI Journal*, Vol. 16, No. 6, Nov.-Dec. 1971, pp. 10-18.
3. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)," American Concrete Institute, Detroit, 1971, 78 pp.

Appendix
Analysis of ultimate load test

At failure the loads on the system were:

Dead load of units = 40 psf
Two layers of a typical span = 80 psf
Concentrated loads as shown on Fig. A1

The top portion of Fig. A1 shows the dimensions and loading arrangement of the system while the lower portion of the illustration gives the shear and moment diagrams for the same system. The span length is 22.75 ft and the width is 13.33 ft.

The system was analyzed as a monolithic unit.

The uniform load on the system is:

$$120 \text{ psf} \times 13.33 = 1600 \text{ lb per ft}$$

and the uniform load at the cutout is:

$$120 \text{ psf} \times 10 = 1200 \text{ lb per ft}$$

From Fig A1, the maximum applied moment at midspan due to all loads is:

139,015 ft-lb (or 1,668,180 in.-lb)

The calculated ultimate moment (M_u) for a single 6HS105A unit is:

463,029 in.-lb (see Fig. A2)

Therefore, the calculated ultimate moment for three equivalent full units is:

$$3 \times 463{,}029 = 1{,}389{,}087 \text{ in.-lb}$$

and hence the system failed at a load greater than the calculated ultimate load.

Design example. Given a roof unit 6 in. deep with a 23-ft span. There is a 40 x 40-in. cutout at midspan.

The total superimposed load on the unit is 38 psf.

It is required to select an appropriate unit.

Since the cutout is 40 x 40 in., let us cut a maximum of 20 in. from one unit.

Thus, the section is now reduced by one-half.

Assume that one unit on each side of the cutout will transfer load from the cutout units.

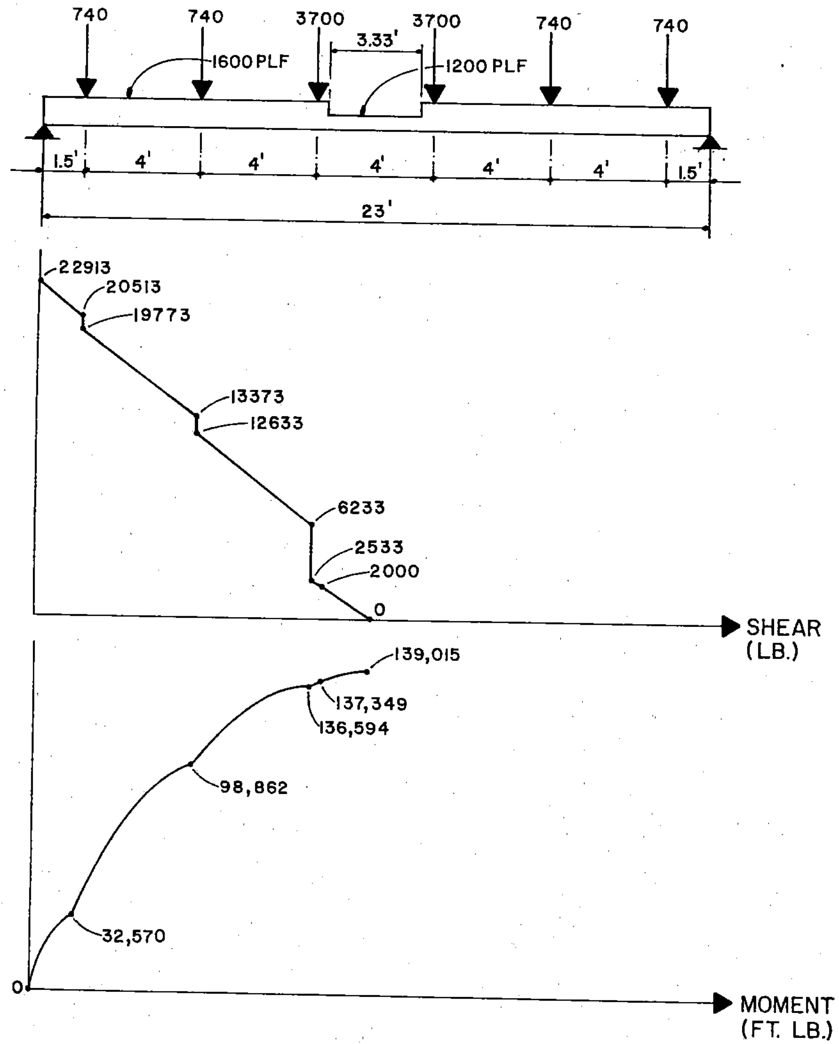

Fig. A1. Load, shear, and moment diagrams

Therefore, the effective section is:

$$40 + 20 + 20 + 40 = 120 \text{ in.}$$

wide at the cutout.

By neglecting the load reduction at the cutout, the dead and superimposed loads are:

Dead load = 40 x 13.33 = 533 lb per ft

Superimposed load = 38 x 13.33 = 507 lb per ft

The moments and stresses at midspan are given by:

$M_{DL} = 1.5 \times 533 \times 23^2 = 423{,}200$ in.-lb

$M_{SL} = 1.5 \times 507 \times 23^2 = 402{,}040$ in.-lb

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6 INCH LIGHTWEIGHT HOUDAILLE SPAN  BARE UNIT
            SAFE SUPERIMPOSED LOAD TABLE
                 POUNDS PER SQ FT

UNIT= 10. 5  0. 0
   EE=     1.350 IN
   EM=     1.350 IN
  FBE=      962. PSI
 FTE=       -2. PSI
RMBE=      290850. IN-LB
RMTE=      488491. IN-LB
  FMB=      962. PSI
 FTM=       -2. PSI
RMBM=      290850. IN-LB
RMTM=      488491. IN-LB
MU =     463029. IN-LB
```

SPAN	LOAD	CONTROL	X-FT	MX IN-LB	MM IN-LB	RMM IN-LB	CAMBR	DL DEFL	NET	SL DEFL
10	522.9	3.	0.00	0.	0.	0.	-.113	.015	-.195	.195
11	426.3	3.	0.00	0.	0.	0.	-.137	.023	-.229	.233
12	352.8	3.	0.00	0.	0.	0.	-.163	.032	-.262	.273
13	295.6	3.	0.00	0.	0.	0.	-.191	.044	-.294	.315
14	250.2	3.	0.00	0.	0.	0.	-.222	.059	-.325	.359
15	213.6	3.	0.00	0.	0.	0.	-.254	.078	-.353	.404
16	183.6	3.	0.00	0.	0.	0.	-.289	.101	-.377	.450
17	158.8	3.	0.00	0.	0.	0.	-.327	.129	-.396	.496
18	137.9	3.	0.00	0.	0.	0.	-.366	.162	-.409	.541
19	120.3	3.	0.00	0.	0.	0.	-.408	.201	-.415	.586
20	105.3	3.	0.00	0.	0.	0.	-.452	.246	-.412	.630
21	92.3	3.	0.00	0.	0.	0.	-.499	.300	-.398	.671
22	81.1	3.	0.00	0.	0.	0.	-.547	.361	-.373	.710
23	71.2	1.	0.00	0.	0.	0.	-.598	.431	-.334	.745
24	62.0	1.	0.00	0.	0.	0.	-.651	.511	-.281	.769
25	53.9	1.	0.00	0.	0.	0.	-.707	.602	-.210	.787
26	46.8	1.	0.00	0.	0.	0.	-.764	.704	-.121	.798
27	40.4	1.	0.00	0.	0.	0.	-.824	.819	-.012	.802
28	34.6	1.	0.00	0.	0.	0.	-.887	.947	.120	.796
29	29.5	1.	0.00	0.	0.	0.	-.951	1.089	.277	.780
30	24.9	1.	0.00	0.	0.	0.	-1.018	1.248	.460	.753
31	20.7	1.	0.00	0.	0.	0.	-1.087	1.422	.671	.713

Fig. A2. Typical computer output

($S_B = 3 \times 209.8$ in.^{3})
($S_T = 3 \times 216.9$ in.^{3})

Load	Stress, psi f_b	f_t
Dead	− 672	650
Superimposed	− 639	618
Total stress	−1311	1268

Select a 6HS105A unit.

From Fig. A2, the prestress is:

$f_b^p = 962$ psi

$f_t^p = -2$ psi

Hence, the net stresses are:

$f_b^n = 962 - 1311 = -349$ psi

which is less than −424 psi (ok).

$f_t^n = 1268 - 2 = 1266$ psi

which is less than 2250 psi (ok).

The required ultimate moment is:

$$M_u \text{ (req.)} = 1.4 \times 423{,}200 + 1.7 \times 402{,}040$$
$$= 1{,}275{,}948 \text{ in.-lb}$$

But, the provided ultimate moment is:

$$M_u \text{ (prov.)} = 3 \times 463{,}029 \text{ in.-lb}$$
$$= 1{,}389{,}087 \text{ in.-lb}$$

Hence, the section is satisfactory.

Discussion of this paper is invited.
Please forward your discussion to PCI Headquarters by January 1, 1973,
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