BRAWLEY POWER PLANT
IN THE
MAY 18, 1940 EARTHQUAKE

PART I

ANALOGOUS STRUCTURE
AND
APPROXIMATE STRESSES

Thesis by

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In Partial Fulfillment
of the Requirements for the
Degree of Master of Science.

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PART I

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A. INTRODUCTION

This choice of subject was the result of a very real interest in earthquake analysis shaped by a number of coincidences. Between late in 1937 and early in 1940 I had designed and inspected the last seven bays of the Brawley Diesel-Electric Power Plant. On May 18, 1940, a severe earthquake (intensity X on Mercalli or Rossi-Forel scales) shook the plant so that permanent set took place in some members. Shortly thereafter I was permitted to show this and other damage to Professor Martel and to discuss some preliminary analysis of the probable stress distribution with him. This thesis and the analyses to follow are an outgrowth of those discussions.

The seismological aspects of the earthquake are very carefully developed in the U. S. Coast and Geodetic Survey publication United States Earthquakes, 1940, Serial No. 647. For details of the mathematics involved, see Analysis of the El Centro Accelerograph Record of the Imperial Valley Earthquake of May 18, 1940, U.S.C.G.S. Report MSS-9. This second report is abstracted on pages 58 through 69 of the first publication mentioned above. For my purpose, the important information is that the maximum horizontal acceleration was approximately 0.36g in a north-westerly direction. Also, the maximum components along and transverse to the building were approximately

0.32g and 0.20g respectively.

In spite of the miles-deep alluvium of the Imperial Valley, a sharp break appeared at the surface along the 40-mile fault with a maximum horizontal displacement of nearly 15 feet. The N 40° W direction of the fault does not correspond too closely to the direction of maximum intensity because later oscillations seemed to line up with the north-south direction of the valley rather than with the fault. The valley is just a huge bowl of alluvium, so reflection of waves from its walls must have been an important factor.

The Brawley Power Plant lies 22 miles north along a N 15° W line from the epicenter and it appears to have been in an intensity IX region. For example, many frame houses were damaged, but every case that we examined had been set on "cripples" which toppled. Most of the damage resulted from falling off of the cripples, but of course the toppling action avoided the peaks of seismic loading.

Brick buildings suffered throughout the valley, and two outstanding examples in the town of Imperial were reduced to absolutely unrecognizeable heaps of loose brick. However, much of the blame for the weakness of brick can be laid to construction methods inadequate for hot dry climates. Bricklayers cannot be made to soak bricks before placing them, so water is drawn out of the mortar with great resultant loss of strength. All brick fractures observed were in the bond, and it seemed that every brick building lost at least a parapet wall.

Adobe displayed the same brittleness as the overdry mortar. The Brawley city hall and fire station lost large sections of their walls by simple crumbling away which allowed other slabs to fall and break. The main cracks were at 45° in the north-south walls, indicating that the heavy tile roof got its major acceleration in that direction. The most intriguing illustration is that of the WPA-built adobe wall around the ball park. About 7 feet high by 22 inches wide, the east-west wall tumbled while the north-south wall stood. Obviously something between 0.2g and 0.3g was necessary to tip over such a wall.

The outstanding examples of reinforced-concrete structures were the Dunlack and Planters' hotels in Brawley. The Dunlack had been advertised as the valley's only earthquake-proof hotel since an earlier quake had cracked interior partitions and dislodged plaster at the Planters' without disturbing the Dunlack. Both buildings are rigid blocks with numerous show windows and openings in the ground floor, and as before the Planters' suffered only superficial damage. The Dunlack however must have hit a resonant vibration because the upper three stories pounded the ground floor until the concrete spalled off parts of the outside columns and bowed the vertical reinforcing rods outward. The vertical one-inch rods had been tied with only three-sixteenths rods at about four-inch spacing. The grand stairway was torn loose from the second floor and the top step was a foot higher than the second floor level. The whole sight gave this designer pause at the possible destructiveness of resonance in an earthquake.

One steel structure of interest was a steel water tower at the Brawley city water works. Although perfectly symmetrical it was twisted about its vertical axis and the diagonal-bracing rods had been stretched. More important however was our framed-steel power-plant building. The plant stretched some of its rods, so we know the extremes of deflection. Also, I have all of the details of analysis and construction of this building. Therefore, I have determined to make the "Brawley Power Plant in the May 18, 1940 Earthquake" the subject of considerable graduate study.

This thesis, known as Part I, determines rather carefully an "Analogous Structure" which will be used in later calculations of stress and deformation. The care taken at this stage is deemed necessary if results are to be compared significantly with the effects observed at the plant. Maximum deflections under a 0.36g static load will be calculated quite accurately in Part II; and if these are appreciably lower than the observed deflections it will be assumed that some dynamic build-up was experienced. In the event of such a difference it is planned to investigate resonant frequencies for Part III.

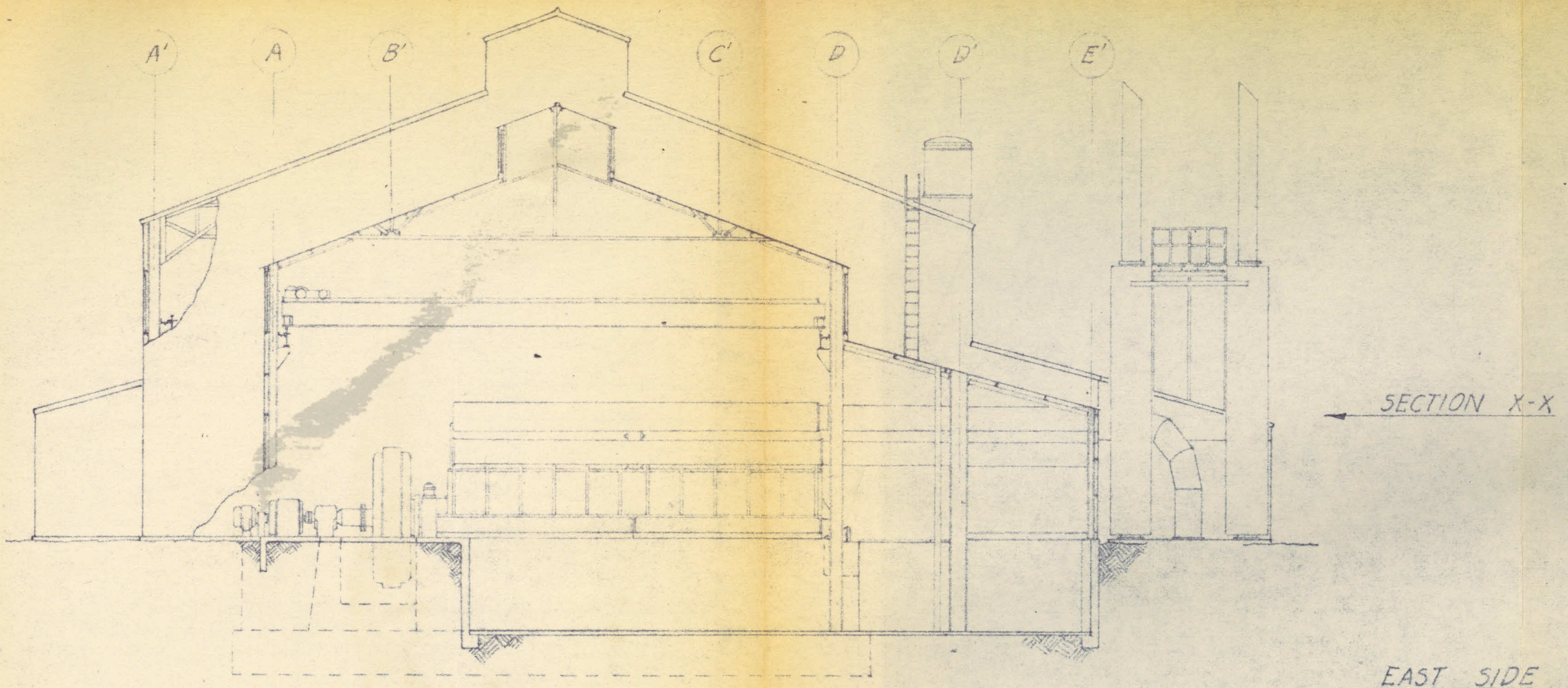
The complication of this preliminary work is the result of the way in which the building grew. The first five bays were built with columns fixed only at the base and with diagonal bracing supplying the greater part of the stiffness. With the 1938 addition to the plant and with other additions to be anticipated, it appeared that the transverse stiffness obtained from cross-braced end-walls was being lost as the building lengthened. Therefore, my first addition has columns fixed top-and-bottom with bents stressed to take the design loads without benefit of the diaphragm action of the bracing in the plane of the lower chord of the truss. This same philosophy was followed when the 1939 addition called for a much broader and higher bent to handle the larger engine-generator installations. The tie between the last three bays and the older part of the building is superficial but just stiff enough to pass troublesome loads between the two sections. The following "Calculations" lead to a "Conclusions" section in which the analagous structure is presented and the first simple check of stresses is run.

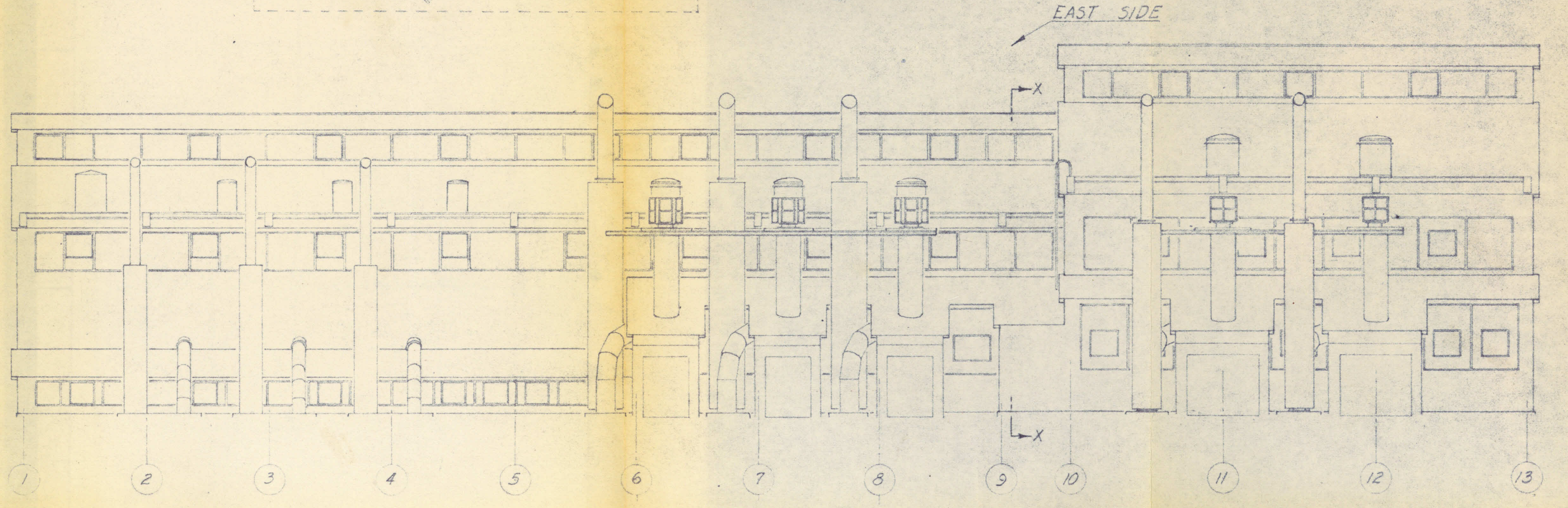
4044-P Feb 8, 1940

Brawley Diesel Plant - Looking south

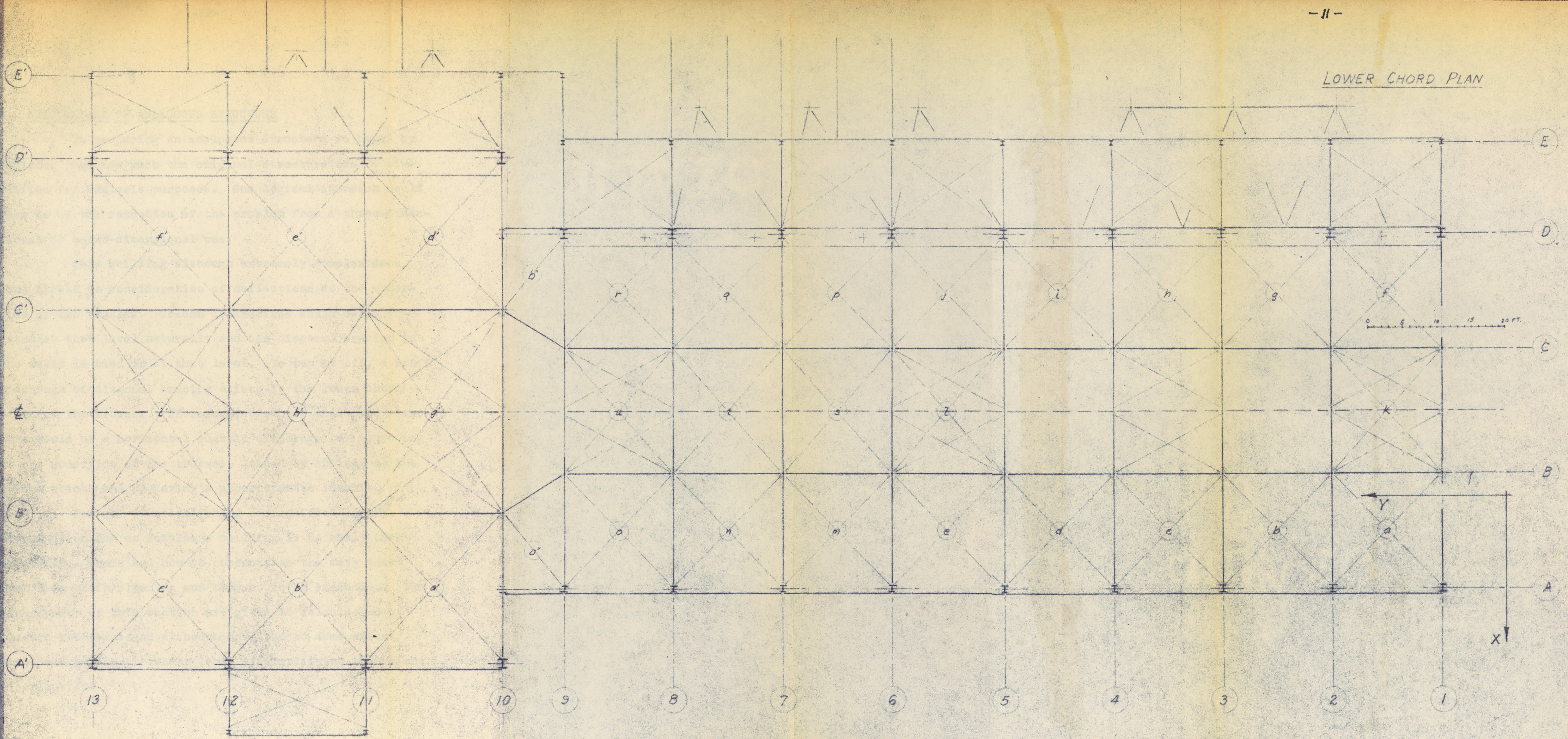
PLANT ELEVATIONS

0 5 10 15 20 FT.





LOWER CHORD PLAN



B. CALCULATION OF ANALOGOUS STRUCTURE

In preparing an analogous structure we begin by deciding just how much the original structure can be simplified for analysis purposes. One logical approach would seem to be the reduction of the problem from a three-dimensional to a two-dimensional one.

This building although extremely complex does lend itself to consideration of deflections at the underside of the trusses. Column deflections would be calculated at that level naturally and the diagonal bracing in the walls is tied in at that level. To top it off, a stiff diaphragm of diagonal bracing exists in the lower chord plane of the trusses. Therefore, the best analogous structure would be a horizontal elastic diaphragm corresponding to the underside of the trusses, loaded by springs at the column points and by having a proportionate inertia.

The job of establishing a simplified weight distribution for so complex a building is no small task by itself. Where and how to concentrate the mass took much thought in planning and many hours in execution. The other parts of this section are given to calculation of elastic constants and although involved to some extent they pale before the sheer labor of simplifying the weight distribution.

1. WEIGHT DISTRIBUTION

The elementary process of finding truss weights and centers of gravity as well as running weights for roofs and walls has been relegated to the appendix section, D. There will be found the size of members and justification for the weights which follow.

Diaphragm framing is divided into three panels per bay, so distributed weights were considered concentrated at their gravitational centers and then apportioned to one of the main panel points. Since the building above the diaphragm is symmetrical about the longitudinal centerline, half of such weight per bent was assigned to each side pair of panel points for similarity of moments as well as of total weight.

Wall weights are carried into the diaphragm system by the columns. Actually, the wall load is divided between the diaphragm and the ground on something like a direct ratio based on height of the elemental weight. A straight-line variation has been used here, i.e., a weight two-thirds of the way up a column is simulated by a weight two-thirds as large concentrated at the column panel point. End walls had to be apportioned on a basis combining the two methods given above.

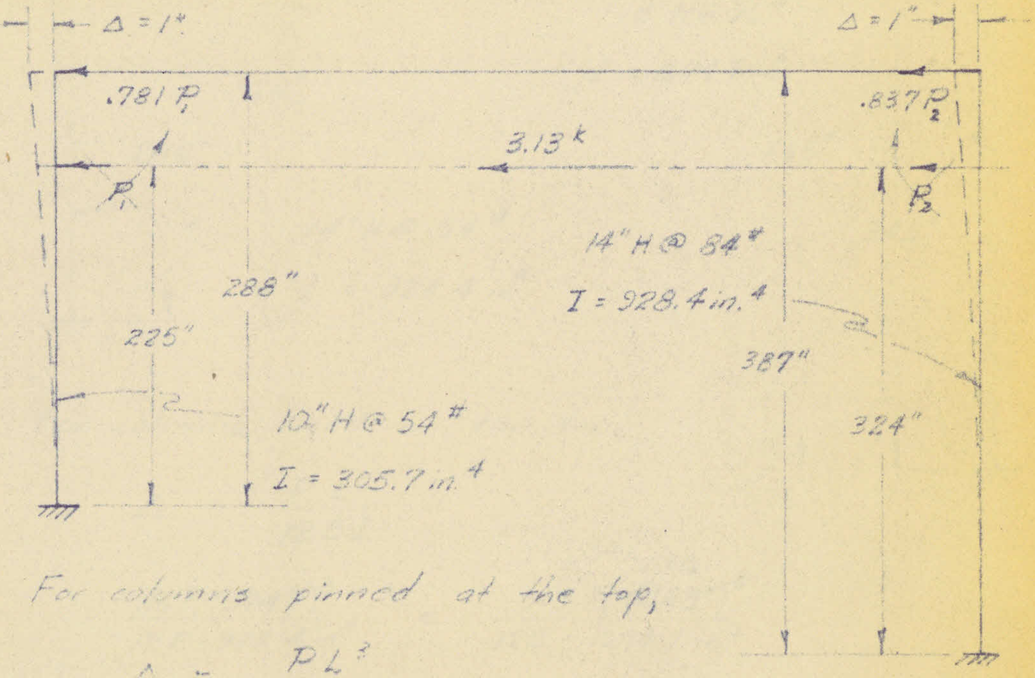
The crane weights at bents #2, 12 and 13 had to be handled differently because of the different heights of the columns at either end of the same bents. Since the cranes were relatively free to roll north-and-south, they are considered as acting only across the building and at the bents where they were located during the earthquake. The equivalent weights for the cranes are calculated on the next two pages and the following sheets give the weight distribution and final concentration at panel points for each bent.

CRANE LOADS

BENT No. 2:

South Crane Wt. = 10 kips

Wt. to Bent #2 = $10^k \cdot \frac{5'}{16'} = 3.13^k$



For columns pinned at the top,

$$\Delta = \frac{PL^3}{3EI}$$

$$\frac{.781 P_1 (288'')^3}{3E \cdot 305.7 in^4} = \frac{.837 P_2 \overset{2.414}{(387'')^3}}{3E \times 928.4 in.^4}$$

$$3.13^k = P_1 + P_2 = \left(1 + \overset{.853}{\frac{.837}{.781} \cdot \frac{305.7}{928.4} \cdot 2.414}\right) P_2$$

$P_2 = \frac{3.13}{1.853} = 1.69^k$; $.837 P_2 = 1.414$

$P_1 = 3.13 - 1.69 = 1.44^k$; $.781 P_1 = \underline{1.125}$

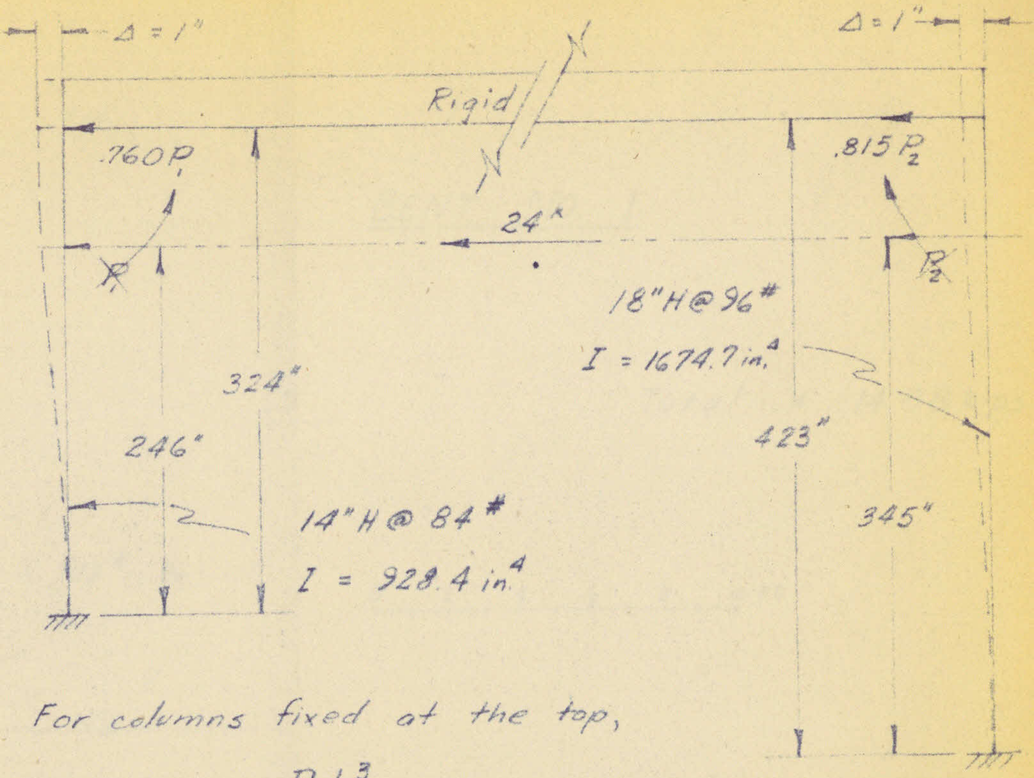
$\underline{\underline{2.54 \text{ kips}}}$ total

BENTS No. 12 & 13:

North Crane Wt. = 48 kips

Wts. to Bents

#12 & 13 = $48^k \cdot \frac{10'}{20'} = 24.0^k$



For columns fixed at the top,

$$\Delta = \frac{PL^3}{12EI}$$

$$\frac{.76P_1\cancel{(324'')^3}}{12E\cdot 928.4\text{ in.}^4} = \frac{.815P_2\overset{2.216}{\cancel{(423'')^3}}}{12E\cdot 1674.7\text{ in.}^4}$$

$$24^k = P_1 + P_2 = \left(1 + \overset{1.318}{\cancel{\frac{.815}{.76}\cdot\frac{928.4}{1674.7}\,2.216}}\right)P_2$$

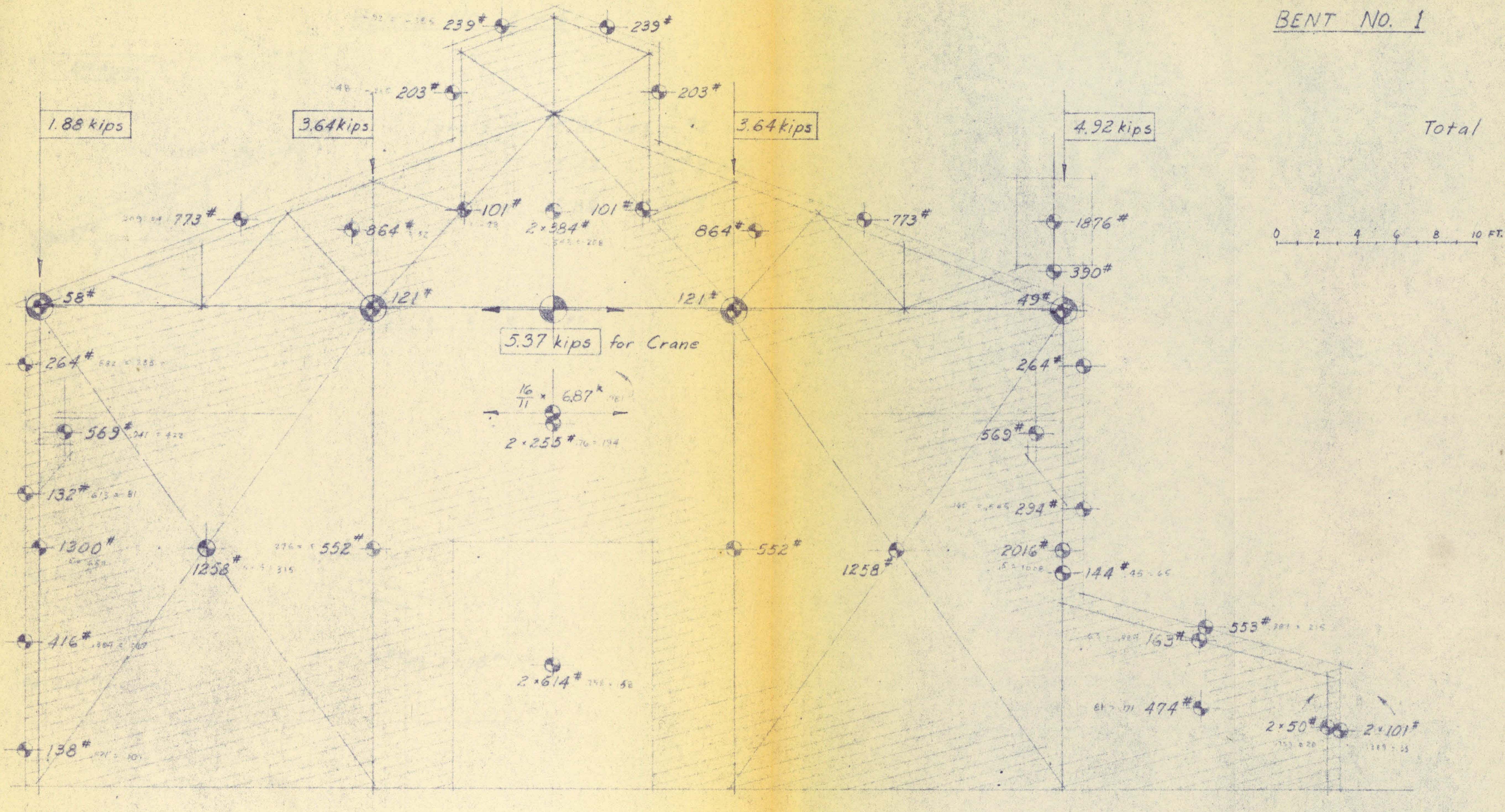
$$P_2 = \frac{24^k}{2.318} = 10.36^k; \qquad .815P_2 = 8.45$$

$$P_1 = 24 - 10.36 = 13.64^k; \qquad .76P_1 = \underline{10.38}$$

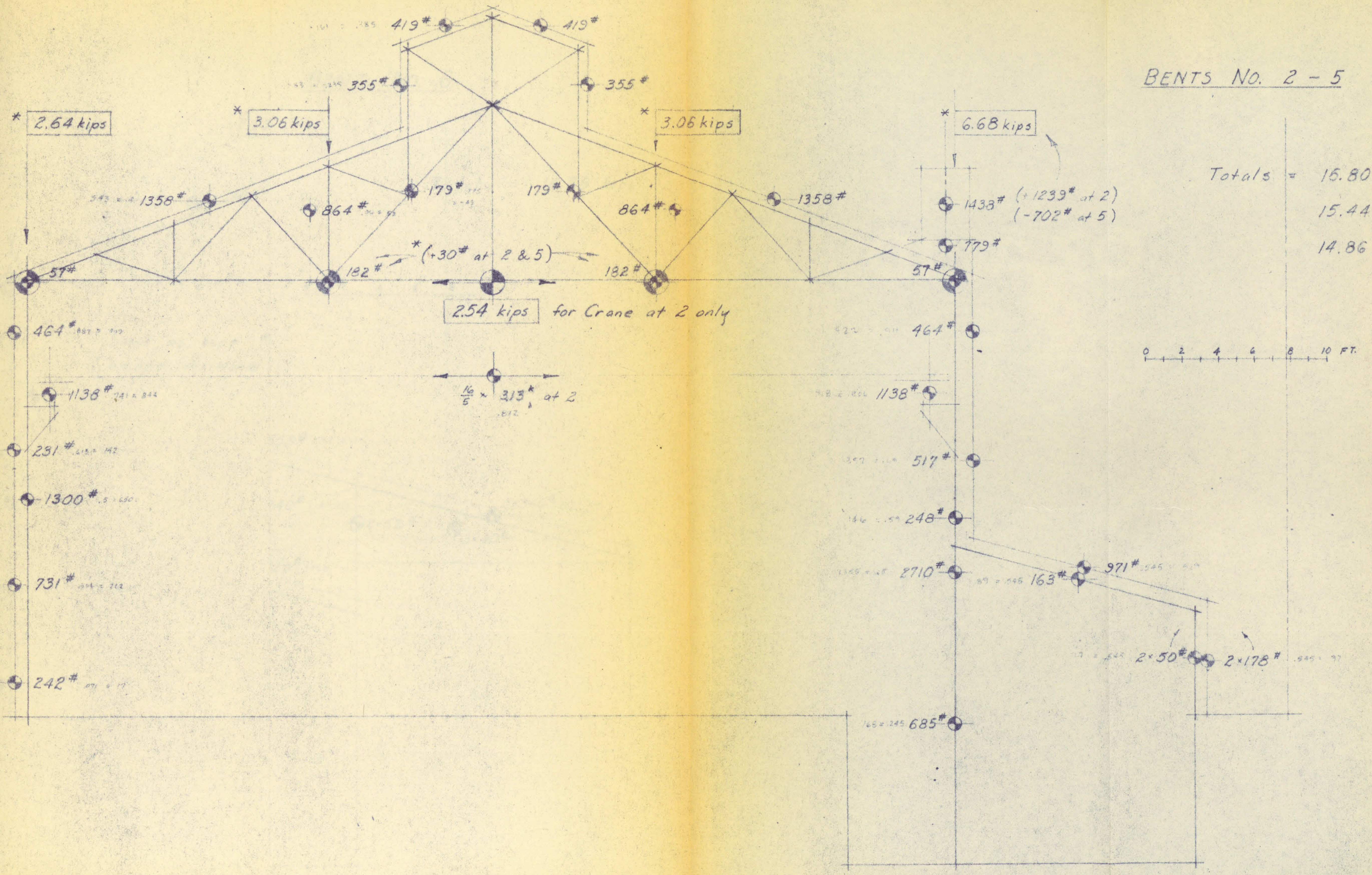
<u>18.8 kips total</u>

BENT NO. 1

Total = 14.08 kips



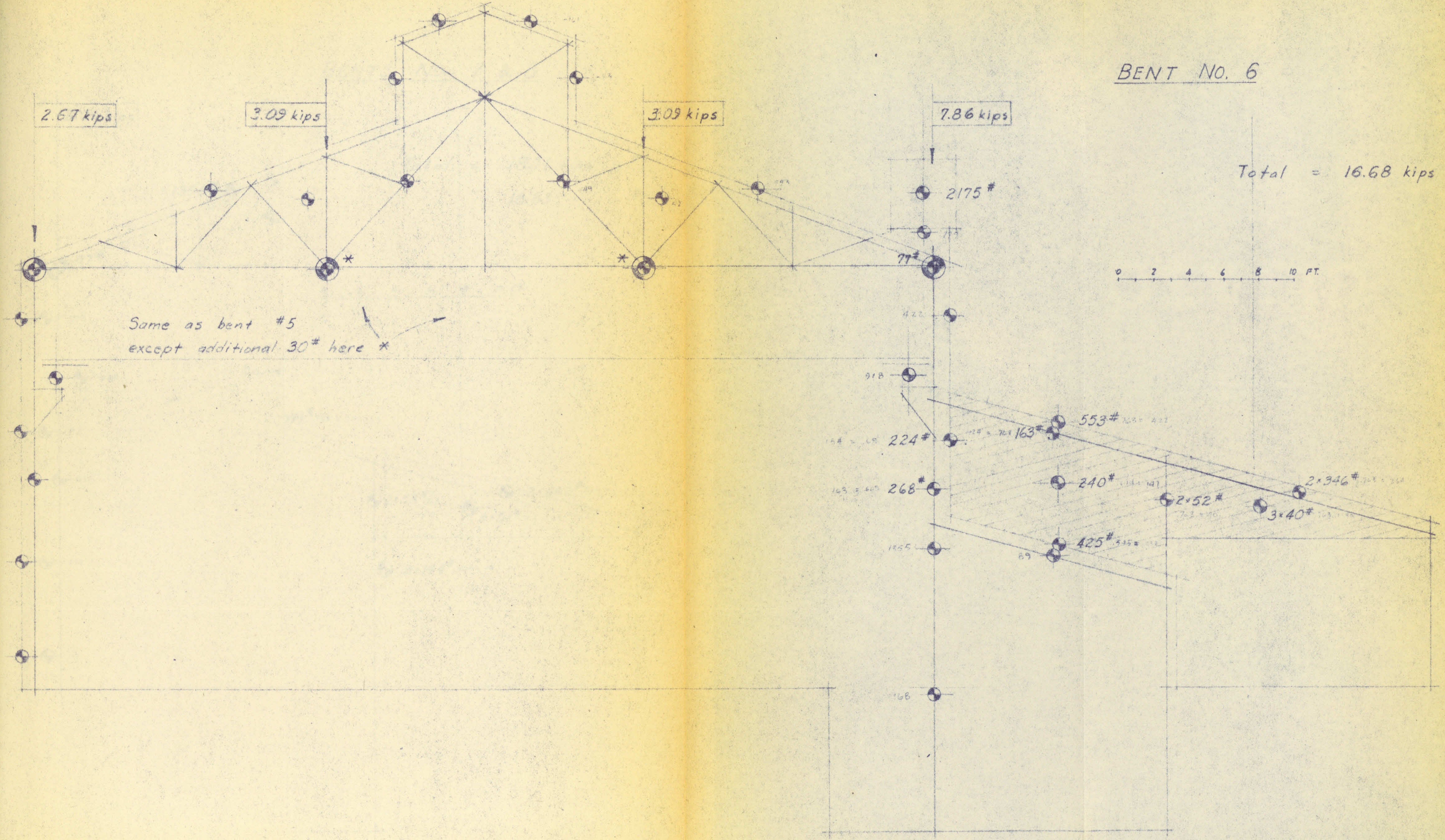
BENTS NO. 2 - 5



Totals = 15.80 kips — #2
15.44 " — #3 & 4
14.86 " — #5

BENT No. 6

Total = 16.68 kips

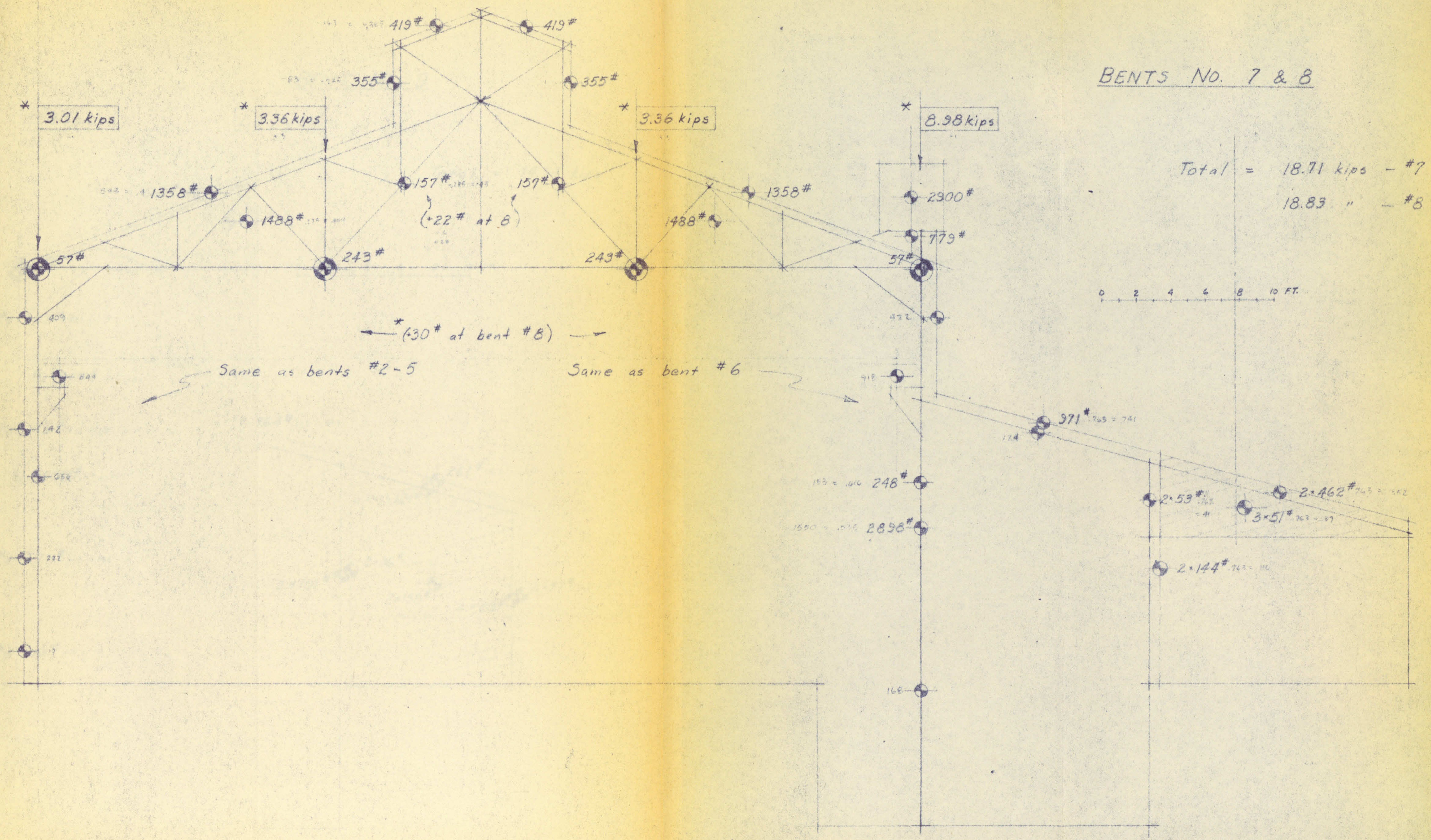


Same as bent #5
except additional 30# here *

BENTS NO. 7 & 8

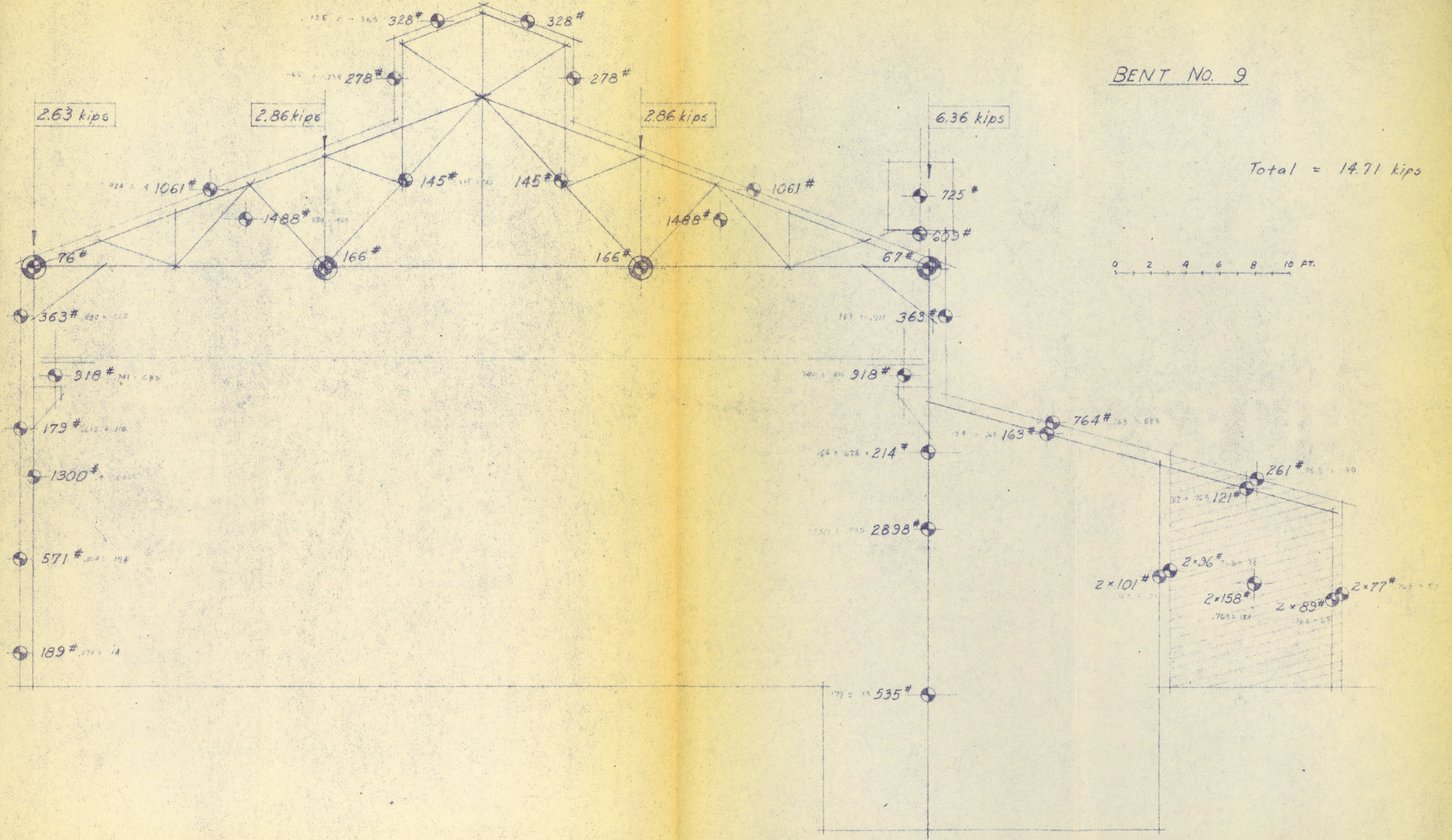
Total = 18.71 kips - #7

18.83 " - #8



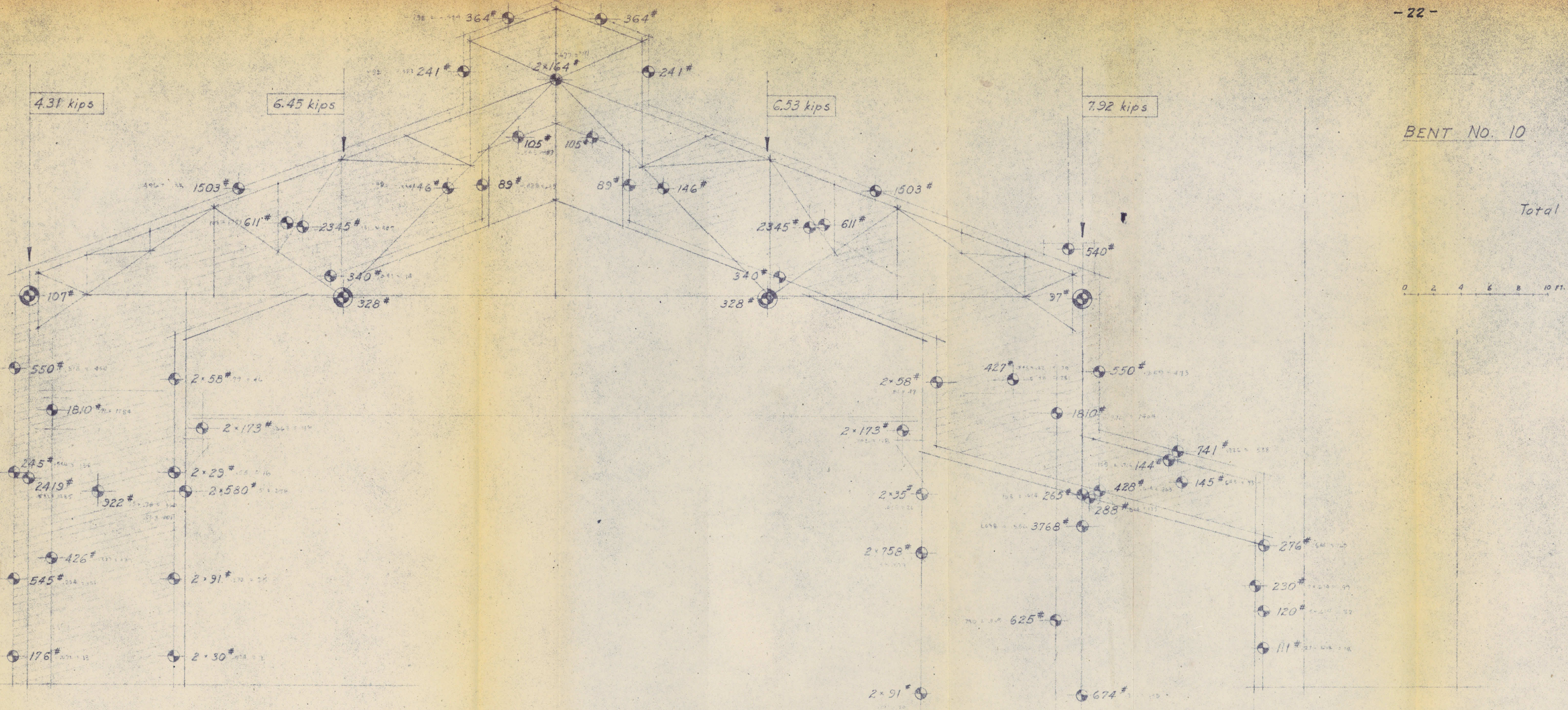
BENT NO. 9

Total = 14.71 kips



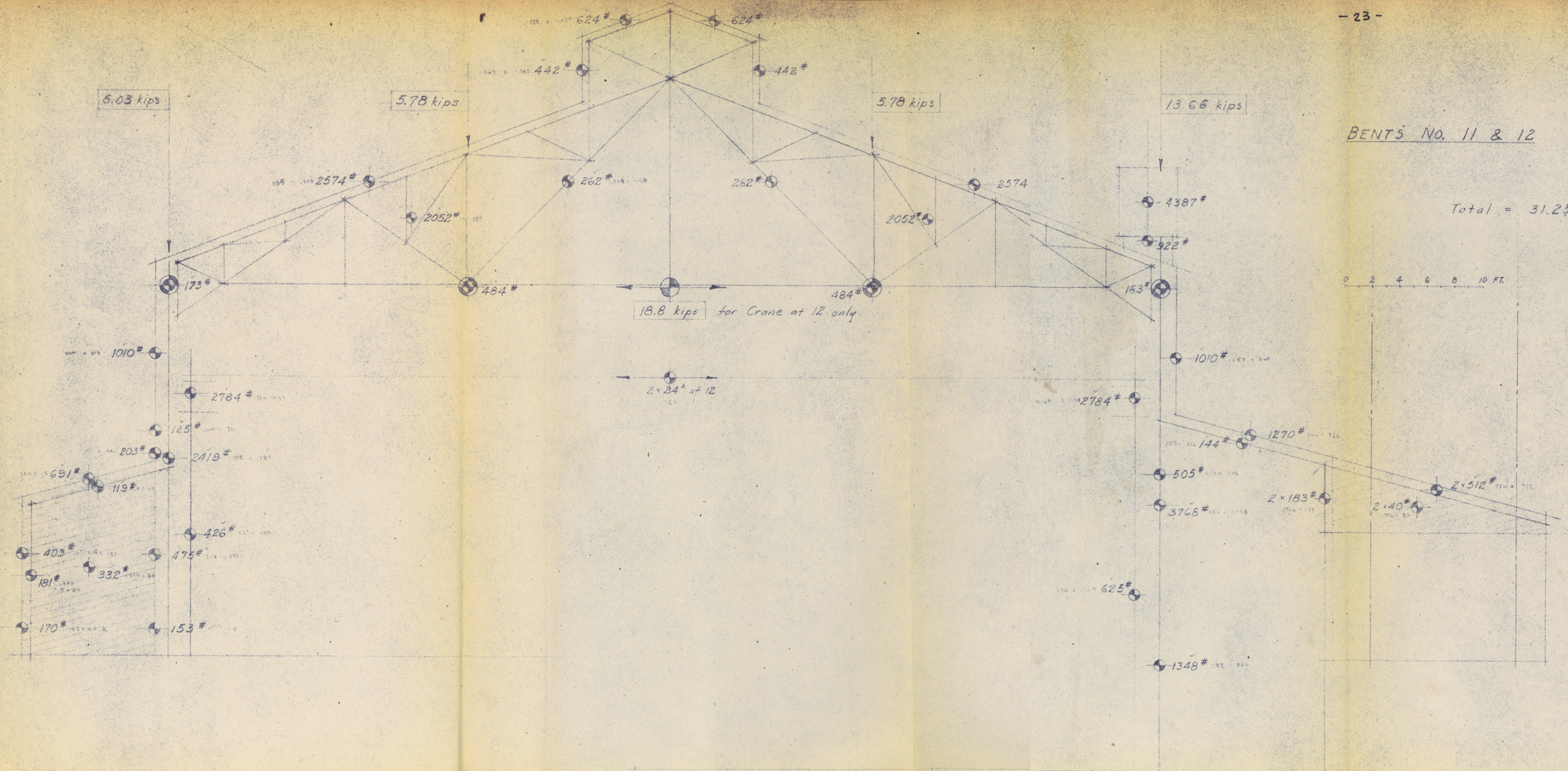
BENT NO. 10

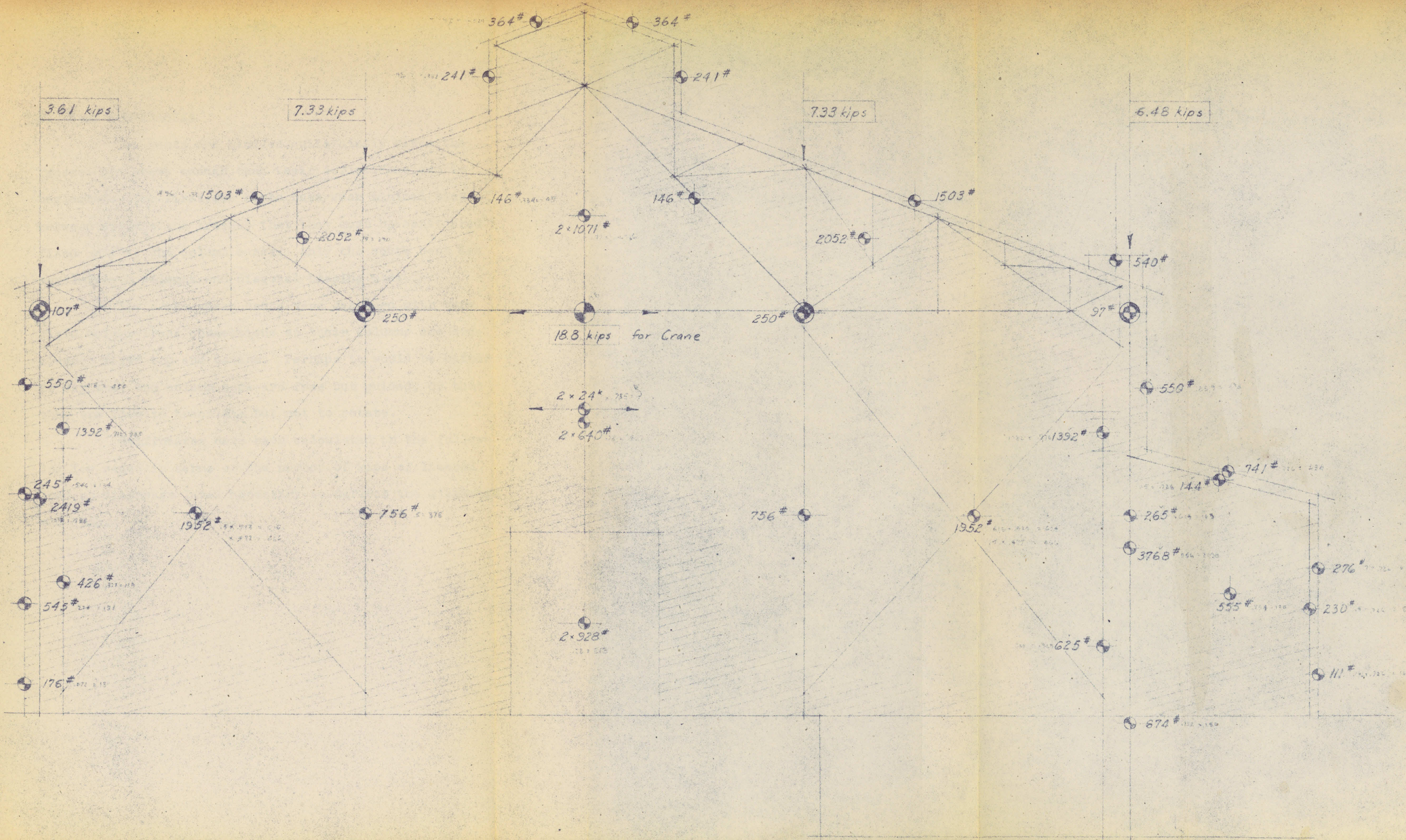
Total = 25.21 kips



BENTS No. 11 & 12

Total = 31.25 kips





BENT NO. 13

Total = 24.75 kips

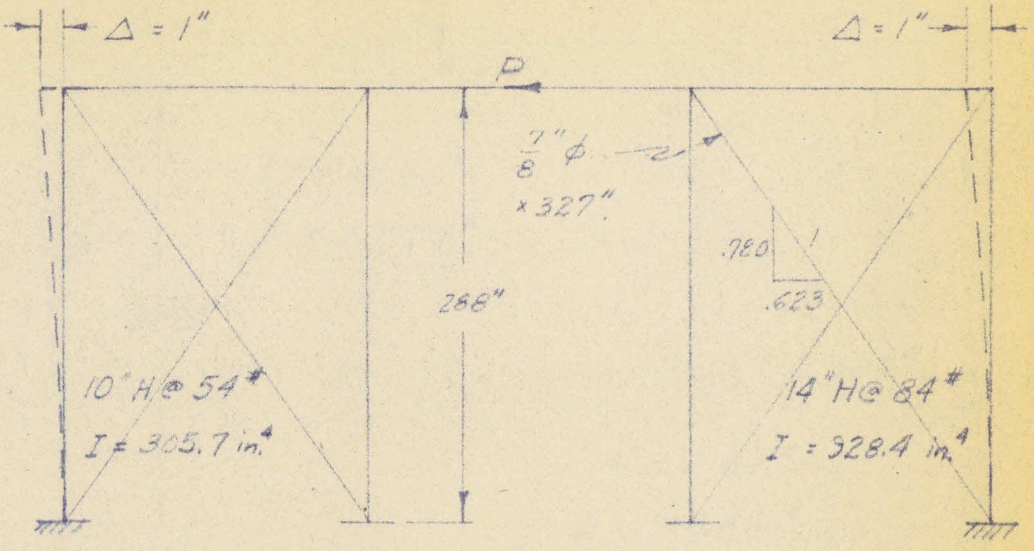
2. BENT STIFFNESSES

The bents are handled quite simply since the trusses are heavy enough that their deflections can be neglected. The first six bents have such minimal ties between columns and trusses that they must be considered fixed only at the column bases. Only the south bent has equal-length columns and diagonal bracing rods.

The north seven bents were designed with very rigid column-truss connections so their columns are considered fixed top and bottom. Perhaps it would be better to say that the column tops are free but guided, or that they are free to translate but not to rotate.

Stiffnesses have been calculated on the following few pages in terms of the number of kips of lateral load at diaphragm level necessary to deflect the diaphragm one inch.

BENT No. 1

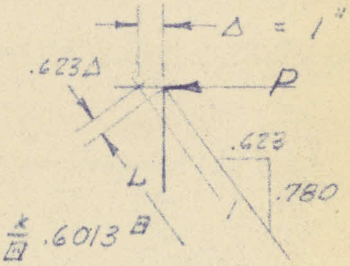


Columns pinned at the top:

$$\left(P/\Delta\right)_A = \frac{3EI}{L^3} = \frac{\overset{87}{\cancel{3\times 29\times 10^3}}\,\frac{k}{\square''}\;305.7\,in^4}{\underset{23,890}{(288'')^3}} = 1.11\ k/''$$

$$\left(P/\Delta\right)_D = \frac{87\times 928.4}{23,890} = 3.38\ k/''$$

Diagonals:



$$\frac{P}{.623} = \frac{.623\,\Delta}{L}\,EA$$

$$P/\Delta = \frac{(.623)^2}{327''}\times 29\times 10^3\,\frac{k}{\square''}\,.6013\,^{\square''}$$

$$= 20.73\ k/''$$

Total Stiffness = 1.11 + 3.38 + 2 × 20.73 = <u>46.0 kips/in.</u>

BENTS No. 2-6 (like #7-9 except)

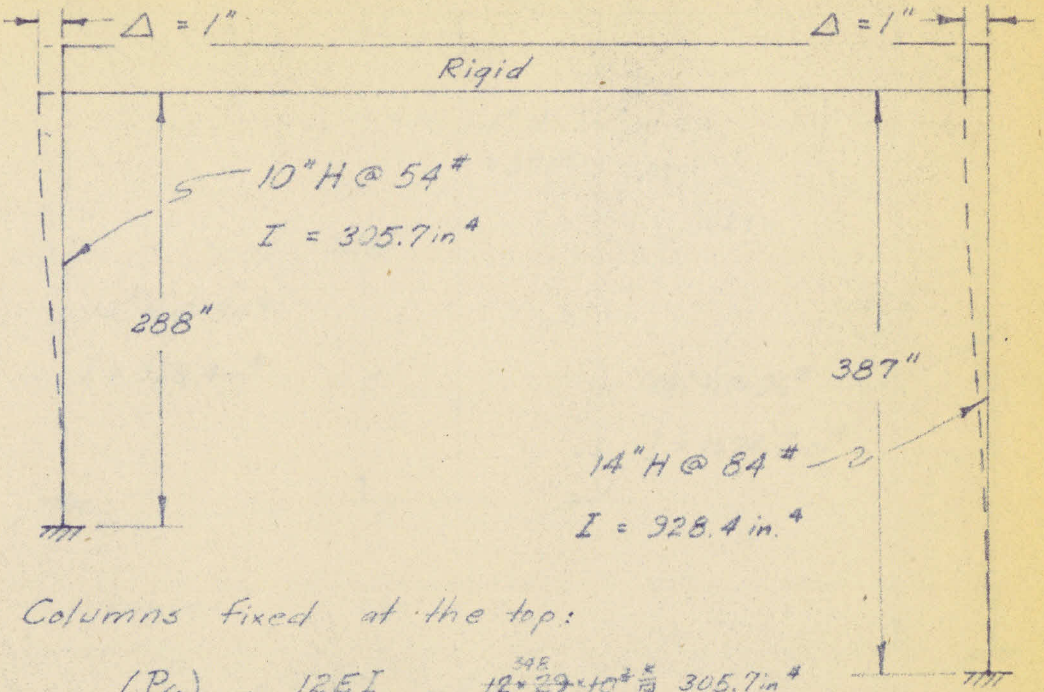
Columns pinned at the top:

$$\left(P/\Delta\right)_A = 1.11\ k/''$$

$$\left(P/\Delta\right)_D = \frac{\overset{87}{\cancel{3\times 29\times 10^3}}\,\frac{k}{\square''}\;928.4\,in^4}{\underset{57,960}{(389'')^3}} = 1.39\ k/''$$

Total Stiffness = 1.11 + 1.39 = <u>2.5 kips/in.</u>

BENTS No. 7-9



Columns fixed at the top:

$$\left(P/\Delta\right)_A = \frac{12EI}{L^3} = \frac{\overset{348}{\cancel{12\times29}}\times10^{3}\frac{k}{\square''}\ 305.7\text{in}^4}{\underset{23,890}{(288'')^3}} = 4.45\ {}^{k}/_{''}$$

$$\left(P/\Delta\right)_D = \frac{348\times928.4}{57,960} = 5.57\ {}^{k}/_{''}$$

Total Stiffness = 4.45 + 5.57 = <u>10.0 kips / in.</u>

BENTS No. 10-12 (like #13 but no diagonals)

Columns fixed at the top:

$$\left(P/\Delta\right)_A = \frac{348\times10^3\times928.4}{\underset{34,010}{(324)^3}} = 9.90\ {}^{k}/_{''}$$

$$\left(P/\Delta\right)_D = \frac{348\times10^3\times1624.7}{\underset{75,690}{(423)^3}} = 7.71\ {}^{k}/_{''}$$

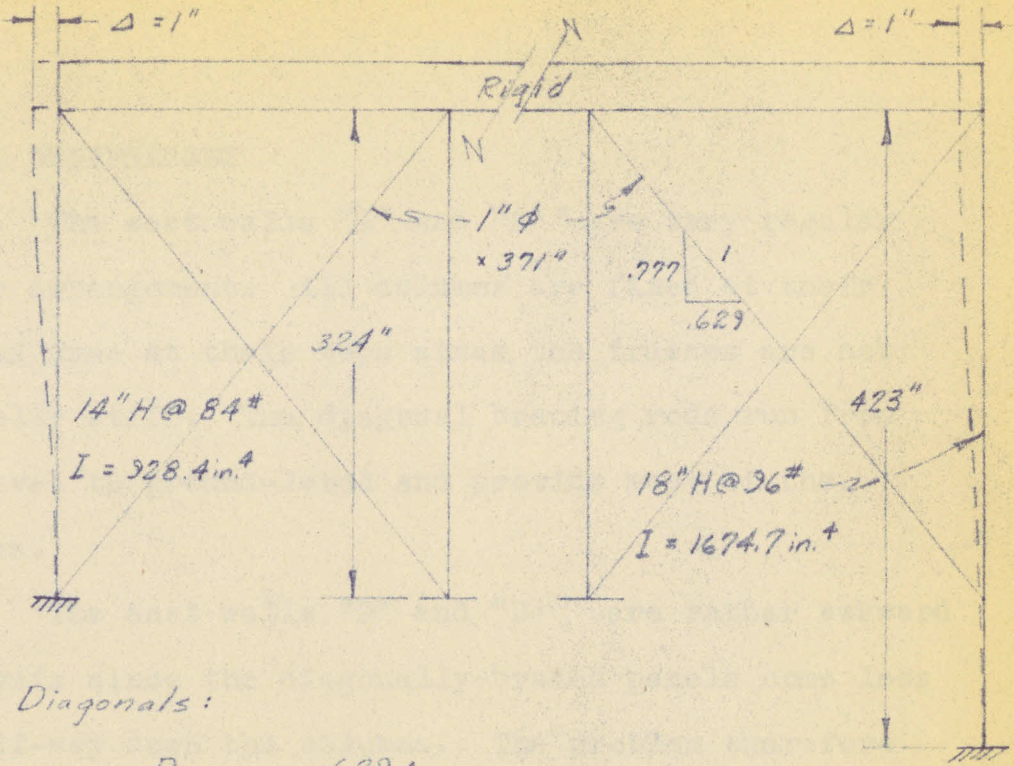
Total Stiffness = 9.90 + 7.71 = <u>17.6 kips / in.</u>

BENT No. 13

Columns fixed at the top:

$$\left(P/\Delta\right)_A = 9.90\ {}^{k}/_{''}$$

$$\left(P/\Delta\right)_D = 7.71\ {}^{k}/_{''}$$



Diagonals:

$$\frac{P}{.629} = \frac{.629\Delta}{L} EA$$

$$P/\Delta = \frac{(.629)^2}{371"} \, 29\times10^3 \frac{k}{\square"} \, .7854^{\square"} = 26.73 \, {}^{k}/_{"}$$

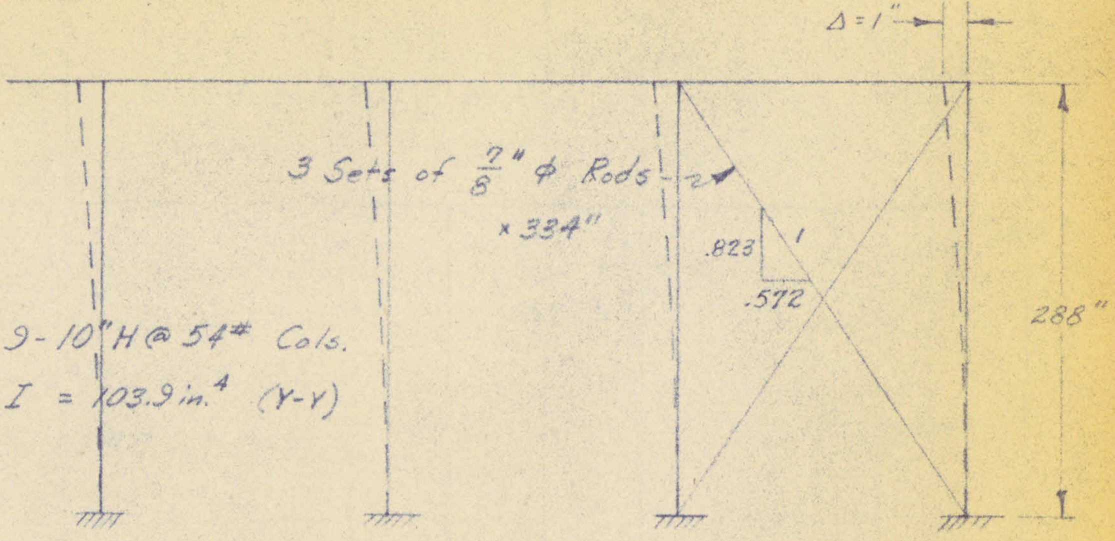
Total Stiffness = 9.90 + 7.71 + 2·26.73 = <u>44.3 kips/in.</u>

3. WALL STIFFNESSES

The west walls "A" and "A'" are very regular in their arrangement. All columns are fixed at their bases and free at their tops since the trusses are not torsionally stiff. The diagonal bracing rods run from truss-level to ground-level and provide most of the stiffness.

The east walls "D" and "D'" are rather awkward of analysis since the diagonally-braced panels come less than half-way down the columns. The problem therefore involves unknown loads and deflections at two levels on the columns. Wall "D" has two more unknowns because its south column is shorter than the rest.

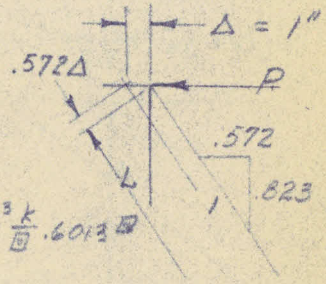
WALL "A"



Columns:

$$\left(P/\Delta\right) = \frac{3EI}{L^3} = \frac{\overset{87}{\cancel{3\times 2}}9\times 10^3 \frac{k}{\square}\ 103.9\ \text{in}^4}{\underset{23,890}{(\cancel{288''})^3}} = 0.38\ {}^{k}/_{"}$$

Diagonals:



$$\frac{P}{.572} = \frac{.572\,\Delta}{L}\,EA$$

$$\left(P/\Delta\right) = \frac{(.572)^2}{334''}\ 29\times 10^3 \frac{k}{\square}\ .6013\,\square$$

$$= 17.09\ {}^{k}/_{"}$$

Total Stiffness = 9 × .38 + 3 × 17.09 = <u>54.7 kips / in.</u>

WALL "D"

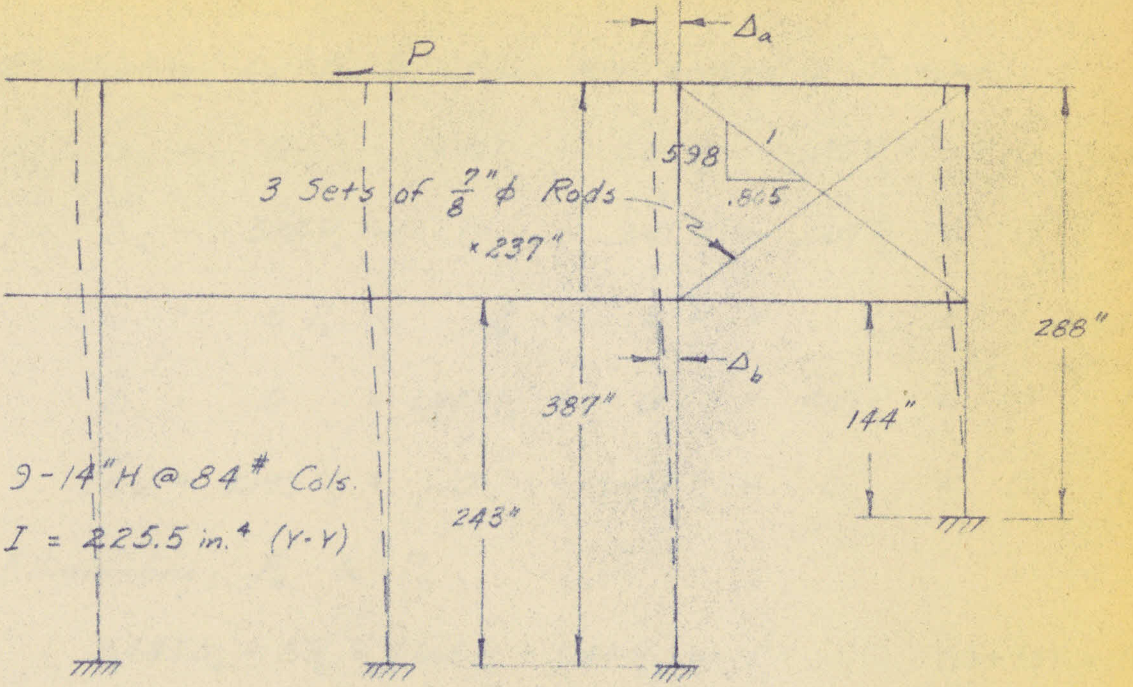
Bent #1:

$$\Delta_a = \frac{P_{a_1} L_1^3}{3EI} + \frac{5P_{b_1} L_1^3}{48\,EI} = \frac{L_1^3}{3EI}\left(P_{a_1} + \frac{5}{16} P_{b_1}\right)$$

$$\Delta_b = \frac{5P_{a_1} L_1^3}{48\,EI} + \frac{P_{b_1} L_1^3}{24\,EI} = \frac{L_1^3}{3EI}\left(\frac{5}{16} P_{a_1} + \frac{1}{8} P_{b_1}\right)$$

Bents #2-9:

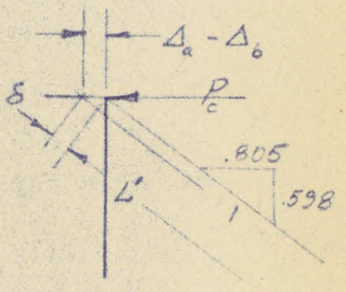
$$\Delta_a = \frac{P_a L^3}{3EI} + \frac{17(27)^2 P_b L^3}{(43)^3 EI} = \frac{L^3}{3EI}\left[P_a + \frac{51\overset{729}{(27)^2}}{\underset{79,507}{(43)^3}} P_b\right]$$



$$\Delta_b = \frac{14{,}464\,P_a L^3}{3(43)^3 EI} + \frac{P_b L^3}{3EI} = \frac{L^3}{3EI}\left[\frac{14{,}464}{(43)^3}P_a + \left(\frac{27}{43}\right)^3 P_b\right]$$

Diagonals :

$$\Delta_a - \Delta_b = \frac{P_c\ L'}{(.805)^2 EA}$$



Additional Conditions:

$$3P_c = .8P_b + P_{b_1}$$

$$P = 8P_a + P_{a_1} + 3P_c = 1^k \text{ say}$$

Substituting Physical Values into the first 5 eqns,

$$\Delta_a = \frac{\overset{23{,}890}{(288'')^3}}{\underset{87}{3\times 29}\times 10^3 \frac{k}{\boxed{\text{in}}}\ 225.5\,\text{in}^4}\left(P_{a_1} + \frac{5}{16}P_{b_1}\right) = 1.308\frac{''}{k}P_{a_1} + .409\frac{''}{k}P_{b_1}$$

$$\Delta_b = .409\,P_{a_1} + .163\,P_{b_1}$$

$$\Delta_a = \frac{\overset{57{,}960}{(387)^3}}{87\times 10^3\times 255.5}\left(P_a + \frac{37179}{79507}P_b\right) = 3.173\,P_a + 1.468\,P_b$$

$$\Delta_b = \frac{31732}{79{,}507}\left(14{,}464\,P_a + 19{,}683\,P_b\right) = .576\,P_a + .785\,P_b$$

$$\Delta_a - \Delta_b = \frac{237''\ P_c}{.648\times .6013\,\boxed{\text{in}}\ 29\times 10^3\frac{k}{\boxed{\text{in}}}} = 0.0210\frac{''}{k}P_c$$

Eliminating Δ_b & P_c from the preceding 7 eqns,

$$\overset{2.445}{\Delta_a} + 0 - .409P_{a_1} - \overset{.137}{\cancel{.056}}P_b - \overset{.416}{\cancel{.170}}P_{b_1} = 0 \qquad (1)$$

$$\overset{13.889}{\Delta_a} - \overset{8}{\cancel{.576}}P_a + 0 - \overset{11.681}{\cancel{.841}}P_b - \overset{.097}{\cancel{.007}}P_{b_1} = 0 \qquad (2)$$

$$0 + 8P_a + P_{a_1} + 8P_b + P_{b_1} = 1 \qquad (3)$$

$$\overset{.765}{\Delta_a} + 0 - 1.308P_{a_1} + 0 - \overset{.313}{\cancel{.409}}P_{b_1} = 0 \qquad (4)$$

$$\overset{2.521}{\Delta_a} - \overset{8}{\cancel{3.173}}P_a + 0 - \overset{3.701}{\cancel{1.468}}P_b + 0 = 0 \qquad (5)$$

Eliminating P_{a_1} & P_a,

$$2.445\Delta_a + 8P_a + 7.863P_b + .584P_{b_1} = 1 \qquad (1)+(3)=(6)$$

$$\overset{9.526}{\cancel{1.680}}\Delta_a - \overset{.776}{\cancel{.137}}P_b - \overset{.584}{\cancel{.103}}P_{b_1} = 0 \qquad (1)-(4)=(7)$$

$$\overset{68.442}{\cancel{11.368}}\Delta_a - \overset{48.044}{\cancel{7.980}}P_b - \overset{.584}{.097}P_{b_1} = 0 \qquad (2)-(5)=(8)$$

$$4.966\Delta_a + 4.162P_b + .584P_{b_1} = 1 \qquad (5)+(6)=(9)$$

Finally, combining (9) with both (7) & (8),

$$\left.\begin{aligned} 14.492\Delta_a - 3.386P_b &= 1 \\ \overset{5.664}{\cancel{23.408}}\Delta_a - \overset{3.386}{\cancel{43.882}}P_b &= \overset{.0772}{\cancel{1}} \end{aligned}\right\} \quad \Delta_a = \frac{.9228}{8.828} = 0.1045''$$

$$\text{Total Stiffness} = \frac{1^k}{.1045''} = \underline{\underline{9.56 \text{ kips/in.}}}$$

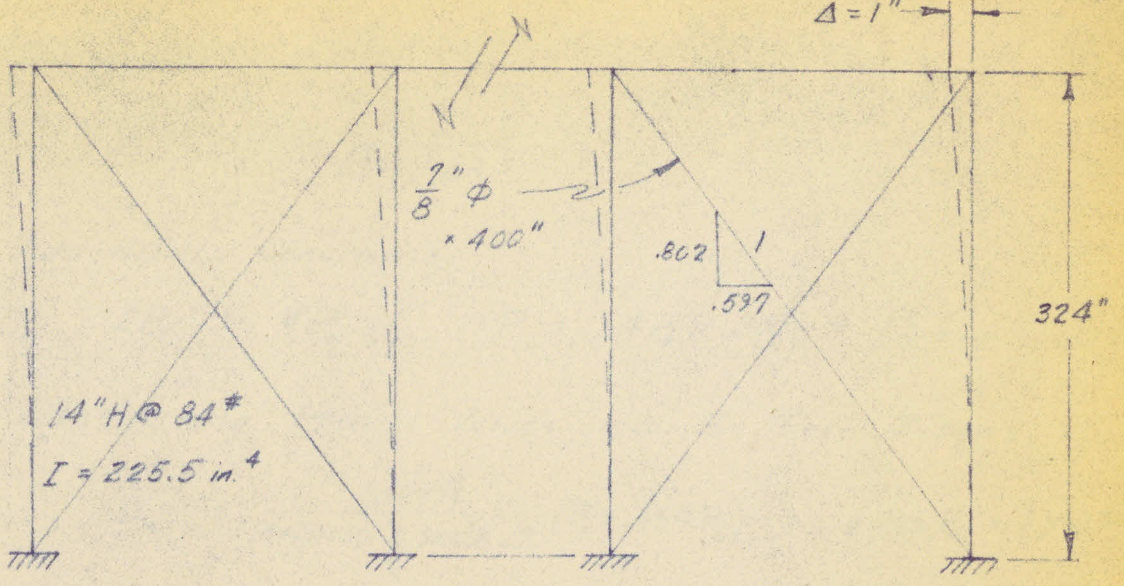
WALL "A'"

Columns:

$$(P/\Delta) = \frac{\overset{87}{\cancel{3\times 29}}\times 10^3 \frac{k}{\square}\ 225.5\text{ in.}^4}{(324'')^3} = 0.58\ ^k/_{''}$$

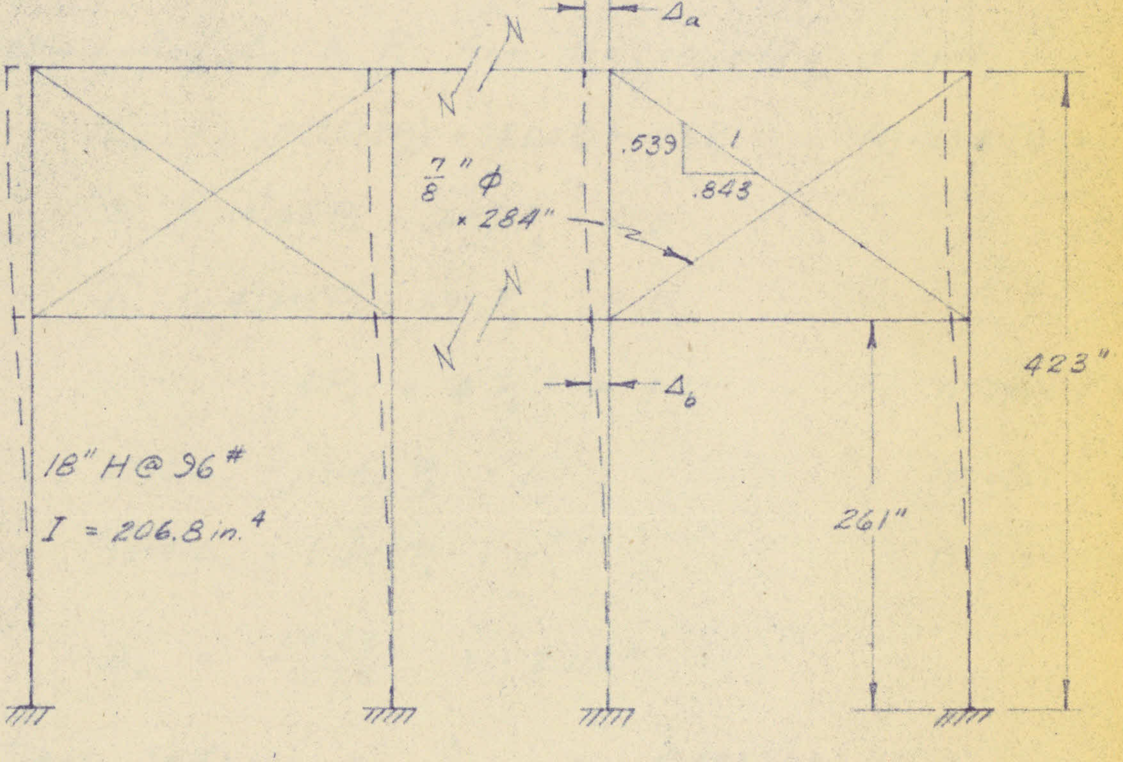
Diagonals:

$$(P/\Delta) = \frac{\overset{356.4}{(.597)^2}}{400''}\cdot 29\times 10^3\frac{k}{\square}\cdot .6013^{\square} = 15.62\ ^k/_{''}$$



Total Stiffness = 4 × .58 + 2 × 15.62 = <u>33.6 kips/in.</u>

WALL "D"



Columns:

$$\Delta_a = \frac{L^3}{3EI}\left[P_a + \frac{56\overset{841}{(29)^2}}{\underset{103,823}{(47)^3}}P_b\right] = \frac{L^3}{3EI}\left(P_a + .454\,P_b\right)$$

$$\Delta_b = \frac{L^3}{3EI}\left[\frac{19,926}{(47)^3}P_a + \left(\frac{29}{47}\right)^3 P_b\right] = \frac{L^3}{3EI}\,4.257\left(.817P_a + P_b\right)$$

Diagonals:

$$\Delta_a - \Delta_b = \frac{P_c\, L'}{(.843)^2 EA}$$

Additional Conditions:

$$2P_c = 4P_b \; ; \qquad P = 4P_a + 2P_c = 1^k \text{ say} \qquad (1 \& 2)$$

Substituting Physical Values into the first 3 eqns,

$$\Delta_a = \frac{\overset{75.687}{(423'')^3}}{\underset{87}{\cancel{3\times 29}}\times 10^3 \frac{k}{\square}\, 206.8\,\text{in}^4}\left(P_a + .454P_b\right) = 4.205P_a + 1.910P_b \quad (3)$$

$$\Delta_b = 4.205 \times 4.257(.817P_a + P_b) = 14.62P_a + 17.90P_b \quad (4)$$

$$\Delta_a - \Delta_b = \frac{284''\ P_c}{.7106 \times 29 \times 10^3 \frac{k}{\square}\, .6013\square} = 0.0229\,\frac{''}{k}\,P_c \quad (5)$$

Eliminating Δ_b & P_c from the preceding 5 eqns,

$$\Delta_a = .0229(2P_b) + 14.62P_a + 17.9P_b \qquad (4)+(5) \& (1) = (6)$$

$$\overset{.2736}{\Delta_a} - \overset{4}{14\cancel{.62}}P_a - \overset{4.909}{\cancel{17.95}}P_b = 0 \qquad \text{do.}$$

$$\overset{.4514}{\Delta_a} - 4\cancel{.205}P_a - \overset{1.817}{\cancel{1.91}}P_b = 0 \qquad (3)$$

$$4P_a + 4P_b = 1 \qquad (1)-(2) = (7)$$

$$.2736\,\Delta_a - .9086\,P_b = 1 \qquad (7)+(6) = (8)$$

$$\overset{.3957}{.95\cancel{14}}\,\Delta_a + \overset{.9086}{\cancel{2.1881}}\,P_b = \overset{.4162}{\cancel{1}} \qquad (7)+(3) = (9)$$

$$\Delta_a = \frac{1.4162}{.6693} = 2.116''$$

$$\text{Total Stiffness} = \frac{1^k}{2.116''} = \underline{0.473 \text{ kips / in.}}$$

4. DIAPHRAGM STIFFNESSES

Stiffness of the diaphragm has been reduced here to a matter of the elongation of individual panel diagonals. This was done in anticipation of final deformations which are not likely to be the same across any one bay or along any one side. Ultimately the relative motion of adjacent bents and longitudinals will have to be correlated with the extension of the acting diagonal.

For the labeling of panels in the calculations which follow, see the "Lower Chord Plan" on page 11 or the "Analogous Structure" on page 39.

PANELS a - j

$$(P/\Delta) = \frac{EA}{L} = \frac{29\times10^{3}\,\frac{k}{\square}\,.6013^{\square}}{260''} = \underline{\underline{67.1 \text{ kips/in.}}}$$

PANELS k & l

$$(P/\Delta) = \frac{29\times10^{3}\times .6013}{273} = \underline{\underline{63.9 \text{ kips/in.}}}$$

PANELS m - r

$$(P/\Delta) = \frac{29\times10^{3}\times .7854}{260} = \underline{\underline{87.8 \text{ kips/in.}}}$$

PANELS s - u

$$(P/\Delta) = \frac{29\times10^{3}\times .7854}{273} = \underline{\underline{83.4 \text{ kips/in.}}}$$

PANELS a' - f'

$$(P/\Delta) = \frac{29\times10^{3}\times .7854}{339} = \underline{\underline{67.2 \text{ kips/in.}}}$$

PANELS g' - l'

$$(P/\Delta) = \frac{29\times10^{3}\times .7854}{407} = \underline{\underline{55.9 \text{ kips/in.}}}$$

PANELS a" & b"

$$(P/\Delta) = \frac{29\times10^{3}\times .7854}{160} = \underline{\underline{142.5 \text{ kips/in.}}}$$

C. CONCLUSIONS

Part I of this report on "Brawley Power Plant in the May 18, 1940 Earthquake" is concluded with presentation of an "Analogous Structure and a Preliminary Stress Analysis". The former merely presents the results calculated in section "B" in one diagram with some discussion. The latter involves some calculations which are included in this section and which only come to a preliminary conclusion as to stresses reached in the earthquake.

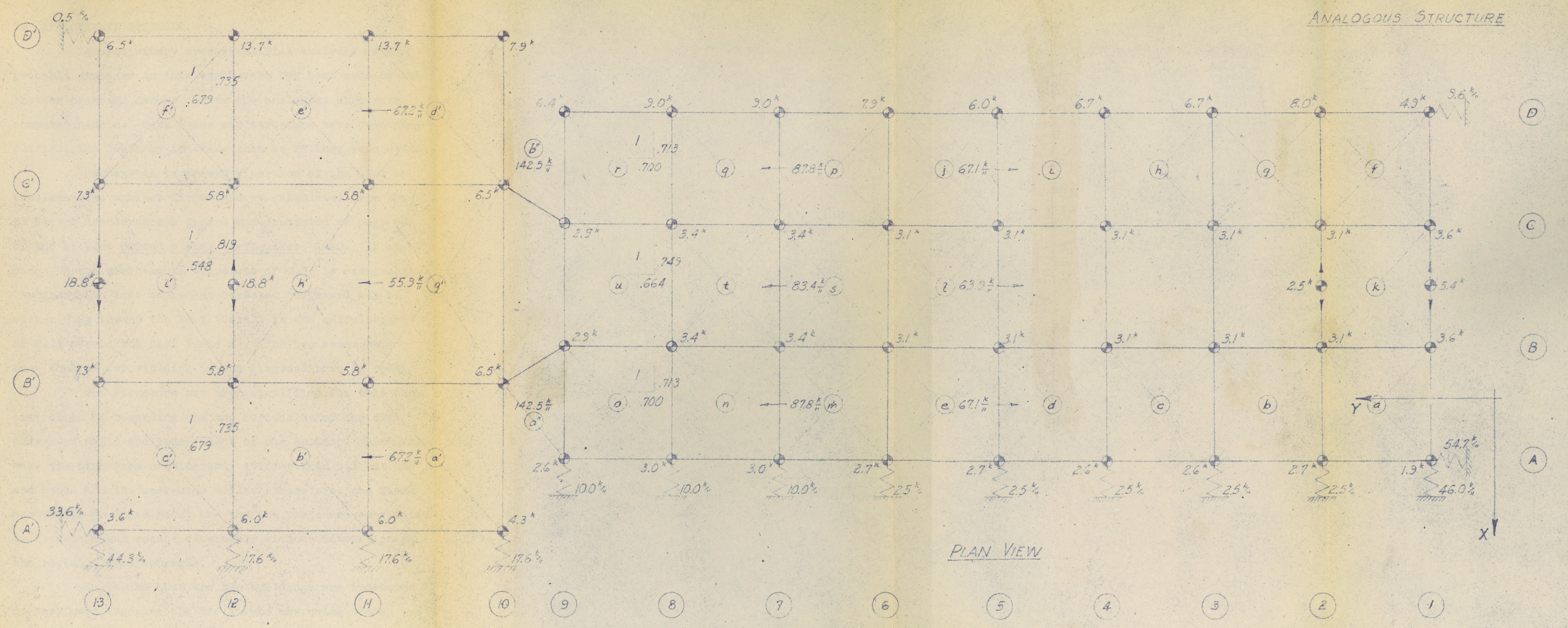
1. ANALOGOUS STRUCTURE

The problem of the entire building's action under horizontal inertia loads has been reduced to the action of a plane array of concentrated weights and springs. The next figure shows this array and gives the calculated weights and spring constants.

One immediately notices the poor tie between the two breadths of building. The single strut and rod on each side causes the two parts to wrack under interaction instead of acting together. Also, the extreme flexibility of the east walls is a poor feature. Since these weaknesses are my own design, I am free to say that they are very poor. Ultimately I hope to make a recommendation to the Imperial Irrigation District of simple repairs which will balance out the strengths of various parts to the best advantage.

Also to be noted are the crane weights which are indicated in the middles of bents #1, 2, 12 and 13. Because the cranes are relatively free to move along their tracks, these weights are considered to act only in the X-direction.

ANALOGOUS STRUCTURE



PLAN VIEW

2. APPROXIMATE STRESSES

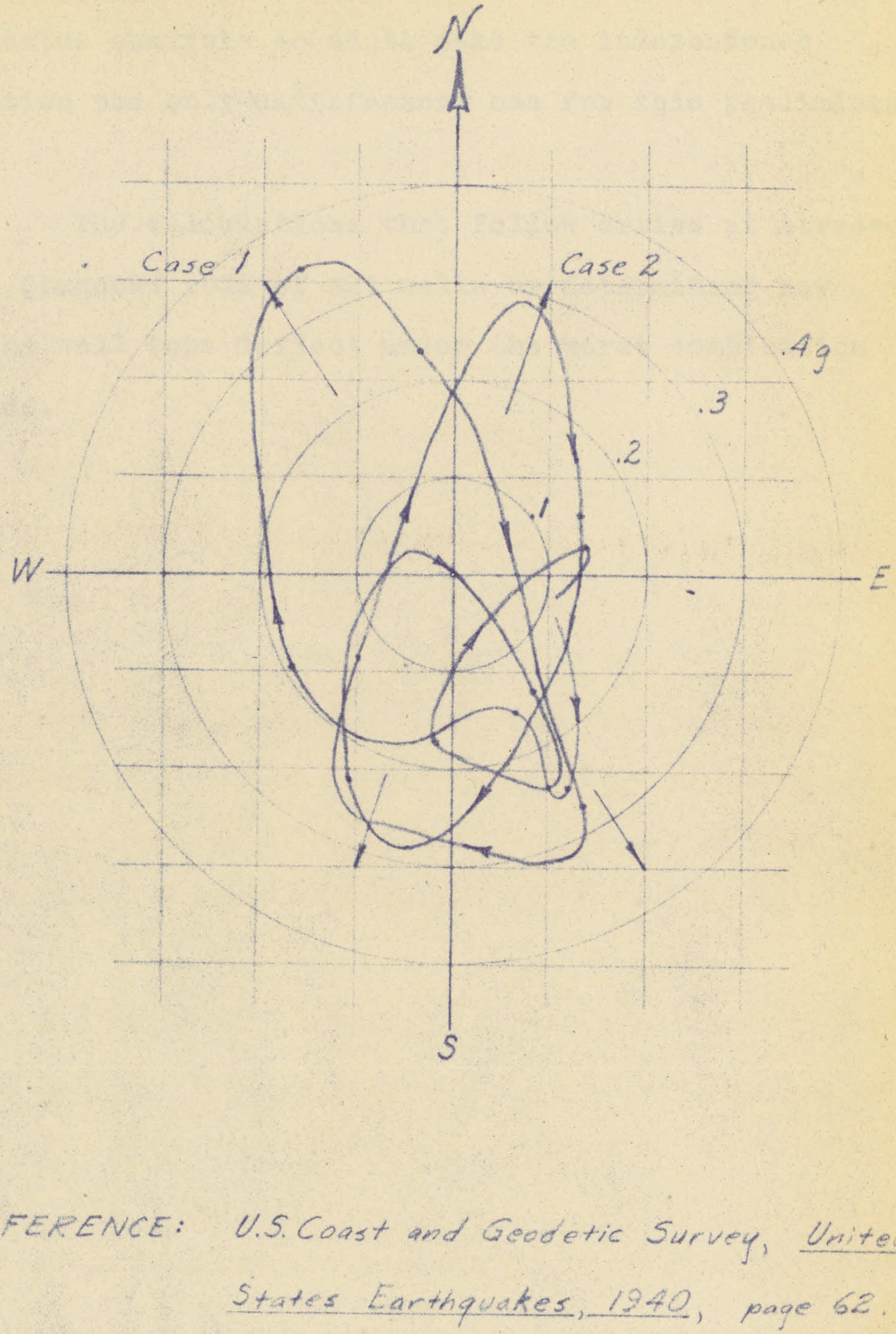
A preliminary over-simplified analysis of the probable stresses in this earthquake has been made to see whether observed damage checks the analogous structure computed herein. Both loads and structure have been simplified, the loads by treating them as if they were static.

On page 41 is traced a portion of the plot of accelerations against directions as calculated from the El Centro accelerograph record and presented on page 62 of the booklet United States Earthquakes, 1940. It appears that a north-south component of 0.3g is reasonable. Combined with this component directed northward may be either 0.2g toward the West (Case 1 in the calculations) or 0.1g toward the East (Case 2). The two cases have been checked for yielding in the diagonal bracing rods.

The structure has been simplified for this consideration by regarding the two rectangles of the diaphragm as rigid and independent of one another. Diagonals make the diaphragm considerably stiffer than all but the end bents and the west-side walls. Therefore, the first assumption is quite good; and the two parts are considered to translate and rotate separately without distortion of the rectangular diaphragms.

To assume that the two buildings are independent is very poor, because the tie struts are quite stocky and

HORIZONTAL ACCELERATIONS



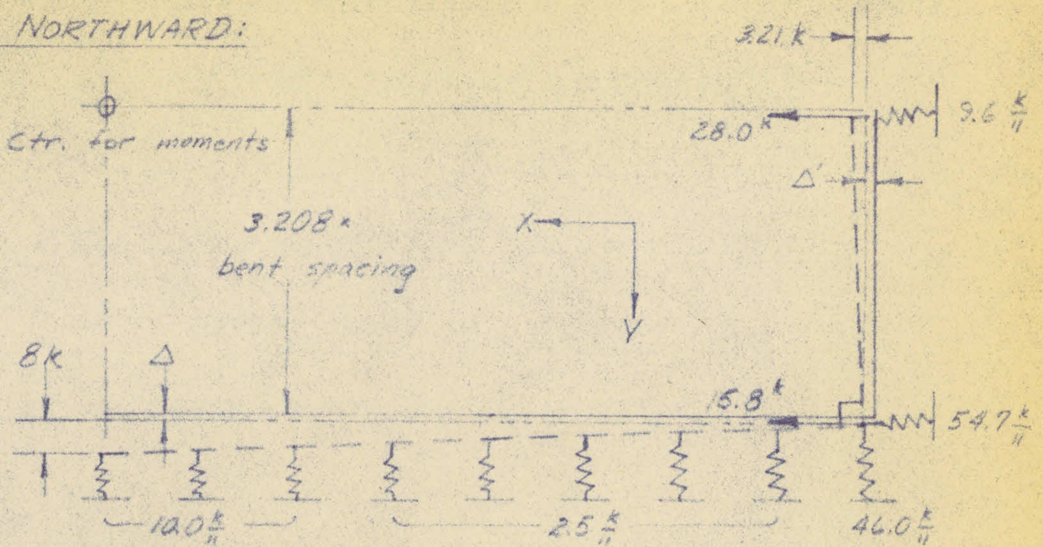
REFERENCE: U.S. Coast and Geodetic Survey, United States Earthquakes, 1940, page 62.

the tension rods are stiff. However, the peculiar tie complicates analysis so as to make the independence assumption the only satisfactory one for this preliminary work.

The calculations that follow arrive at stresses in the diagonal rods of all walls by determining how much the wall tops deflect under the worst combination of loads.

SOUTH SECTION

0.3g NORTHWARD:



$$\Sigma F_x: \quad \overset{30.8}{\cancel{9.6\tfrac{k}{''}\times 3.21}}\,k + \overset{2.08766}{\overset{\cancel{643}}{\cancel{(9.6+54.7)}}}\tfrac{k}{''}\,\Delta' = \overset{1.42708}{\overset{\cancel{43.8}}{\cancel{28.0^k+15.8^k}}} \qquad (1)$$

$$\Sigma F_y: \quad \overset{2.7966}{\overset{\cancel{247.5}}{\left[10(8+7+6)+2.5(5+4+3+2+1)\right]}}\tfrac{k}{''}\,k + \overset{88.5}{\cancel{(3+10+5\times 2.5+46)}}\tfrac{k}{''}\,\Delta = 0 \qquad (2)$$

$$\Sigma M_z: \quad \overset{.7590}{\overset{\cancel{352.5}}{\cancel{\left[10(1\times7+2\times6)+2.5(3\times5+4\times4+5\times3+6\times2+7\times1)\right]}}}\tfrac{k}{''}\,k + \overset{460.5}{\cancel{\left[10(1+2)+2.5(3+4+5+6+7)+46\times8\right]}}\tfrac{k}{''}\,\Delta + \overset{.3813}{\overset{\cancel{175.6}}{\cancel{3.21\times54.7}}}\tfrac{k}{''}\,\Delta' = \overset{.10992}{\overset{50.62}{3.21\times15.8^k}} \qquad (3)$$

$$\overset{.18713}{\cancel{.3813}}\,\Delta' - \cancel{2.0376}\,k = \overset{.05395-}{\cancel{.10992}} \qquad (3)-(2)=(4)$$

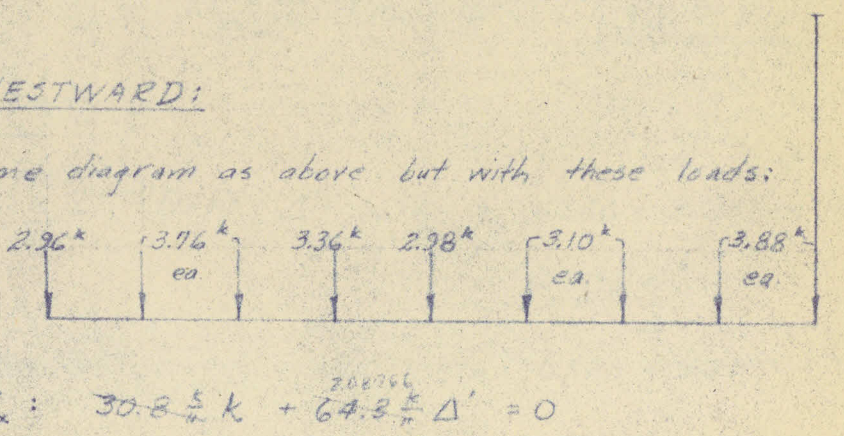
$$2.2748\,\Delta' = 1.4760\,; \quad \Delta' = \underline{\underline{0.6488''}} \qquad (4)+(1)=(5)$$

$$k = \overset{.12140}{\cancel{.1871\times.6488}} - .05395 = \underline{\underline{0.06745''/\text{bent}}} \qquad (4)$$

$$\Delta = -2.7966\times.06745 = \underline{\underline{-0.1886''}} \qquad (2)$$

0.2g WESTWARD:

Same diagram as above but with these loads:



$$\Sigma F_x: \quad \cancel{30.8}\tfrac{k}{''}\,k + \overset{2.08766}{\cancel{64.3}}\tfrac{k}{''}\,\Delta' = 0 \qquad (1)$$

$$\Sigma F_y: \quad \overset{2.7966}{\cancel{247.5}}\tfrac{k}{''}\,k + \cancel{88.5}\tfrac{k}{''}\,\Delta = \overset{.3478}{\overset{\cancel{30.78}}{\cancel{2.96^k+7.52^k+3.36^k+2.98^k+6.2^k+7.76^k}}} \qquad (2)$$

$$\Sigma M_z: \quad \overset{.7590}{\cancel{352.5\tfrac{k}{''}}}\,k + \cancel{460.5\tfrac{k}{''}}\,\Delta + \overset{.3813}{\cancel{125.6\tfrac{k}{''}}}\,\Delta' = \overset{.75434}{\overset{\cancel{118.08}}{\left[\cancel{(1+2)3.76 + 3\times 3.36}\right.}}$$

$$\left.\cancel{+4\times 2.98 + (5+6)\,3.10 + (7+8)\,3.38}\right] \qquad (3)$$

$$\cancel{2.0376}\,k - \overset{.18713}{\cancel{.3813}}\,\Delta' = \overset{.04592}{\cancel{.09356}} \qquad (2)-(3)=(4)$$

$$2.2748\,\Delta' = -.04592\;;\quad \Delta' = \underline{\underline{-0.02018''}} \qquad (4)-(1)=(5)$$

$$k = (-2.088)(-.02018) = \underline{\underline{0.04214\,''/\text{bent}}} \qquad (1)$$

$$\Delta = .3478 - 2.797\times .0421 = \underline{\underline{0.2300''}} \qquad (2)$$

BENT No. 1:

Case 1: Defl., $\Delta = -.1886 + .2300 = 0.0414''$

Case 2: $= -.1886 - \overset{.1150}{\cancel{\tfrac{1}{2}.230}} = \underline{\underline{-0.3036''}}$

Diagonal Rod Stress,

$$S_t = \frac{\delta}{L}E = \frac{\overset{.1891}{\cancel{.623}}\,\Delta}{326''}\,29000\,\tfrac{k}{\square} = \boxed{16.8\text{ ksi}}$$

WALL "A":

Case 1: Defl., $\Delta' = .6488 - .0202 = 0.6286''$

Case 2: $= .6488 + \tfrac{1}{2}.0202 = \underline{\underline{0.6589''}}$

Diagonal Rod Stress,

$$S_t = \frac{\overset{.3769}{\cancel{.592}}\,\Delta}{334''}\,29000\,\tfrac{k}{\square} = \boxed{32.7\text{ ksi}}$$

WALL "D":

Case 1: Defl., $\Delta' + 3.21k = .6286 + \overset{.3518}{\cancel{3.21}}\overset{.1096}{\cancel{(.06745 + .04214)}}$

$= \underline{\underline{0.9804''}}$

Case 2: $= .6589 + \overset{.1402}{\cancel{3.21}}\overset{.0437}{\cancel{(.06475 - \tfrac{1}{2}.04214)}}$

$= 0.7991''$

Continuing the stiffness calculation on p. 32,

$$P_b = \frac{\overset{.5144}{\cancel{14.49\times.1045}}-1}{3.386} = 0.1519^{k} \qquad (9)\div(7)$$

$$P_{b_1} = \frac{1-\overset{.51895}{\cancel{4.966\times.1045}}-\overset{.43221}{\cancel{4.162\times.1519}}}{.584} = \frac{-.1511}{.584} = -0.2588^{k} \; (9)$$

$$\Delta_a - \Delta_b = \overset{.00669}{\cancel{0.00699}}\,(\overset{.9564}{\cancel{8\times.1519-.2588}}) = 0.00669''$$

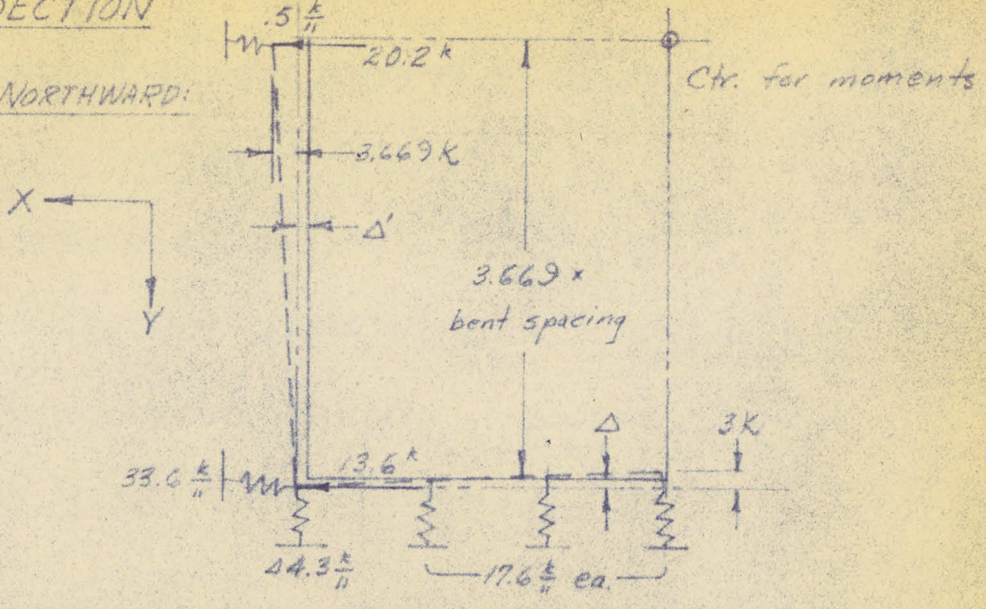
$$S_{t/k} = \frac{\overset{.00538''}{\cancel{.805(\Delta_a-\Delta_b)}}}{237''}\,29{,}000\,\frac{k}{\square} = 0.659\text{ ksi}$$

Diagonal Rod Stress:

$$S_t = S_{t/k}\,\frac{\Delta'+3.21k}{\Delta_{a/k}} = 0.659\,\frac{k}{\square}\,\frac{.9804''}{.1045''} = \boxed{6.2\text{ ksi}}$$

NORTH SECTION

0.3g NORTHWARD:



$\Sigma F_x:\quad \overset{1.885}{.5\frac{k}{''}\times 3.669k} + \overset{18.583}{\cancel{\overset{34.1}{(.5+33.6)}}}\Delta' = \overset{18.420}{\cancel{\overset{33.8}{20.2^k+13.6^k}}}$ (1)

$\Sigma F_y:\quad \overset{1.0875}{\cancel{\overset{105.6}{(1+2+3)17.6\frac{k}{''}}}}k - (\overset{97.1}{\cancel{44.3+3\times17.6}})\frac{k}{''}\Delta = 0$ (2)

$\Sigma M_z:\quad \overset{.3791}{\cancel{\overset{70.4}{17.6\frac{k}{''}(1\times2+2\times1)}}}k - \left[\overset{185.7}{\cancel{44.3\times3+17.6(2+1)}}\right]\frac{k}{''}\Delta + \overset{.6639}{\cancel{\overset{123.28}{3.669\times33.6}}}\frac{k}{''}\Delta'$

$= \overset{.2687}{\cancel{\overset{49.90}{3.669\times13.6^k}}}$ (3)

$-.7084k + \overset{.9372}{\cancel{.6639}}\Delta' = \overset{.3792}{\cancel{.2687}}$ (3)-(2) = (4)

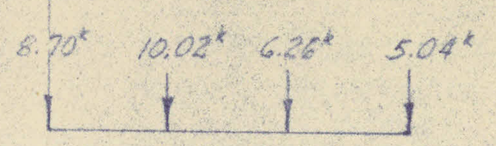
$19.520\Delta' = 18.799\ ;\quad \Delta' = \underline{\underline{0.9631''}}$ (4)+(1) = (5)

$k = .9372\times.9631 - .3792 = \underline{\underline{0.5234''/\text{bent}}}$ (4)

$\Delta = 1.0875\times.5234 = \underline{\underline{0.5692''}}$ (2)

0.2g WESTWARD:

Same diagram as above but with these loads:



$\Sigma F_x:\quad \cancel{1.885}\frac{k}{''}k + \overset{18.583}{\cancel{34.1}}\frac{k}{''}\Delta' = 0$ (1)

$\Sigma F_y:\quad -\overset{1.0875}{\cancel{105.6}}\frac{k}{''}k + \cancel{97.1}\frac{k}{''}\Delta = \overset{.3092}{\overset{30.02}{\cancel{8.70^k+10.02^k+6.26^k+5.04^k}}}$ (2)

$$\Sigma M_z:\ \overset{.3791}{\cancel{-70.4\tfrac{k}{''}}}\,k + \cancel{185.7\tfrac{k}{''}}\,\Delta - \overset{.6639}{\cancel{123.3\tfrac{k}{''}}}\,\Delta' = \overset{.2821}{\overset{\cancel{52.4}}{\cancel{3\times 8.7^{k}}}} +$$

$$2\times 10.02^{k} + 1\times 6.26^{k} \qquad (3)$$

$$\cancel{.7084}\,k - \overset{.9372}{\cancel{.6639}}\,\Delta' = \overset{.03826}{\cancel{-.0292}} \qquad (3)-(2) = (4)$$

$$19.52\,\Delta' = -.03826\ ;\quad \Delta' = \underline{\underline{0.00196''}} \qquad (1)-(4) = (5)$$

$$k = \overset{.00184}{\cancel{.9392\times.00196}} - .03826 = \underline{\underline{-0.03642''/\text{bent}}} \qquad (4)$$

$$\Delta = \cancel{1.8875(-.03642)} + .3092 = \underline{\underline{0.2756''}} \qquad (1)$$

BENT NO. 13:

Case 1: Defl., $\Delta = .5692 + .2756 = \underline{\underline{0.8448''}}$

Case 2: $= .5692 - \frac{1}{2}.2756 = 0.4314''$

Diagonal Rod Stresses:

$$S_t = \frac{\delta}{L}E = \frac{\overset{.5314''}{\cancel{.629\Delta}}}{371''}\,29000\,\frac{k}{\square''} = \boxed{41.5\text{ ksi}}$$

WALL "A'":

Case 1: Defl., $\Delta' = .9631 + .0020 = \underline{\underline{0.9651''}}$

Case 2: $= .9631 - \frac{1}{2}.002 = 0.9611''$

Diagonal Rod Stresses:

$$S_t = \frac{\overset{.5972''}{\cancel{.597\Delta}}}{400''}\,29000\,\frac{k}{\square''} = \boxed{41.8\text{ ksi}}$$

WALL "D'":

Case 1: Defl., $\Delta' + 3.669k = .9651 + \overset{1.7868}{3.669\overset{.4870}{\cancel{(.5234 - .0364)}}}$

$= 2.752''$

Case 2: $= .9651 + \overset{1.9871}{3.669\overset{.5416}{\cancel{(.5234 + \frac{1}{2}.0364)}}}$

$= \underline{\underline{2.952''}}$

Continuing the stiffness calculation on p. 34,

$$P_b = \frac{\overset{.86477}{\cancel{.2736 \times 2.752}} - 1}{.9086} = 0.2149^k \qquad (8)$$

$$\Delta_a - \Delta_b = 0.0229 \times 0.2149 = 0.00492'' \qquad (5)$$

$$S_{t/k} = \frac{\overset{.00415}{\cancel{.843(\Delta_a - \Delta_b)}}}{284''} 29000 \frac{k}{\square} = 0.424 \, ksi$$

Diagonal Rod Stress:

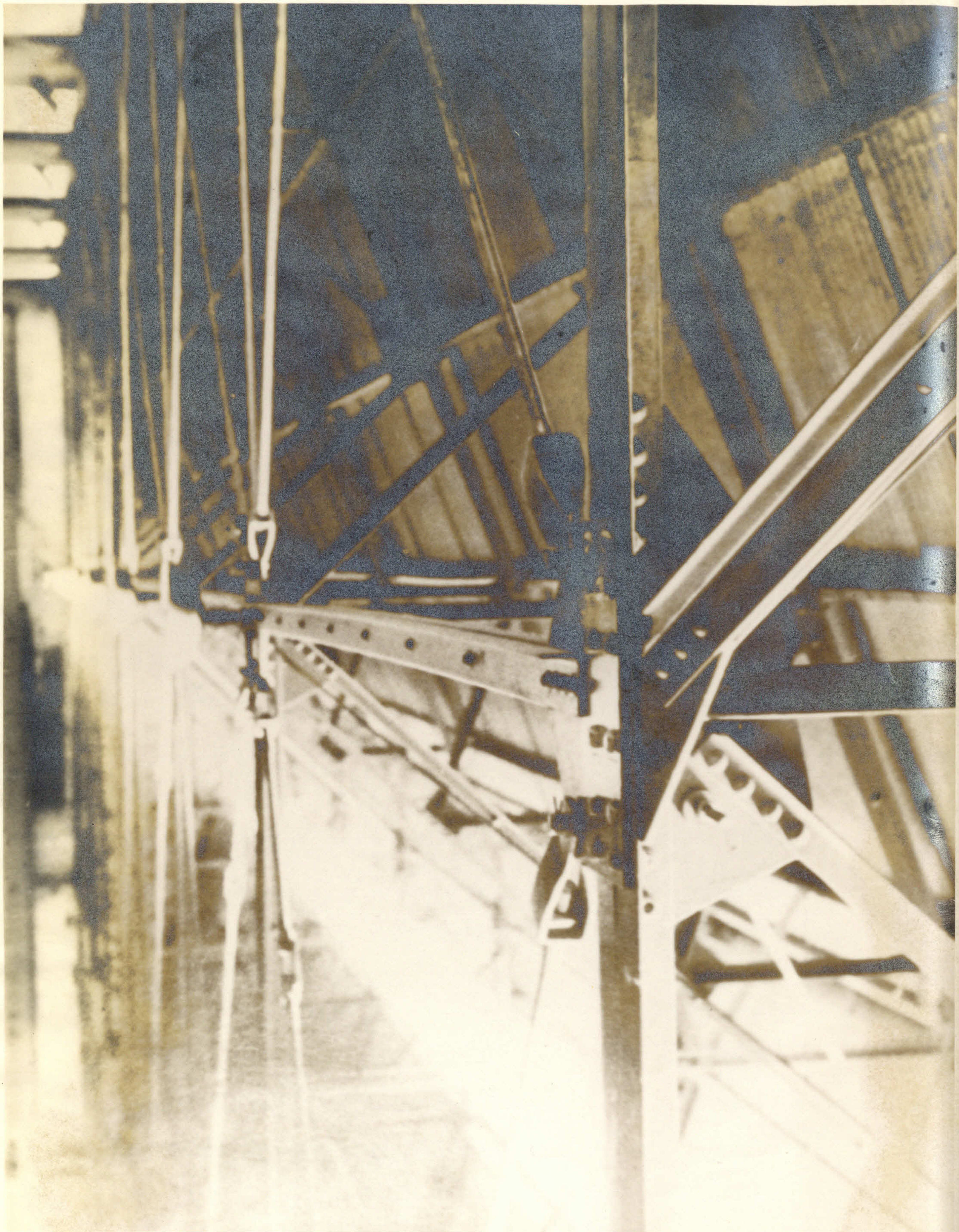
$$S_t = S_{t/k} \frac{\Delta' + 3.669k}{\Delta_{a/k}} = 0.424 \frac{k}{\square} \frac{2.952''}{2.116''} = \boxed{0.59 \, ksi.}$$

The south building appears to have deflected very little, less than an inch for even the weak "D" wall. Although this wall deflected most, the resultant diagonal-rod stress is quite low because of the inadequacy of the rod bracing system. The considerable unbraced length of the columns along this wall accounts for the greater part of the deflection.

The 32,700 p.s.i. stress in the "A" wall rods is just about the yield point but it is not enough for appreciable permanent set. Certainly the calculated deflection of 0.66 inch does not account for the kind of stretching shown in the picture opposite. This picture shows the stretched rods between columns 1 and 2 in the "A" wall.

Other rods were shortened and tightened before pictures could be obtained. However, the other walls in the south building showed no distress as is indicated by the calculations.

The north building, although only one-eighth taller than the south building, deflected considerably more. The 2.95 inch deflection of the "D'" wall looks quite impressive, but resulting rod stress is insignificant because of poor bracing again. The unbraced column length is greater than for wall "D" and the column moment of inertia in the plane of the wall is less.



Since both wall "A'" and bent 13 stretched rods well beyond their yield point, the calculated deflections of just under one inch do not hold true. Calculations were based on the assumption that Hooke's Law holds, so in order to get them correctly we should assume yield stress in the rods and recompute deflections. However, this is enough to show that the reported set of 3 or 4 inches in these rods is within reason.

The larger deflections of the north building did not cause the intermediate tie struts to fail, so the carry-over of this motion to the south building may account for the stretching of rods in the "A" wall.

Still not ruled out is the possibility of a certain amount of dynamic build-up. The considerable but different flexibilities of the two walls along the east side make them particularly suspect. One might expect them to some out-of-phase hammering together with resultant high stresses in the intermediate strut on that side.

The picture opposite looks southward along the strut in the "C" line between bents 8 and 9. The tie strut angling off between bents 9 and 10 is stubbier and was unaffected, but we see here that the strut pictured has been bowed permanently from the compressive load applied to it. This seems to bear out the speculation of the previous paragraph reasonably well.

It is interesting to note that there were no signs of distress around the west strut between buildings. Also, the diagonal rods in bent 1, wall "D" and wall "D'" were not stretched beyond yield. The observed distress checks with these preliminary calculations, so it would appear that the "Analogous Structure" established herein is good enough for more extensive investigation.

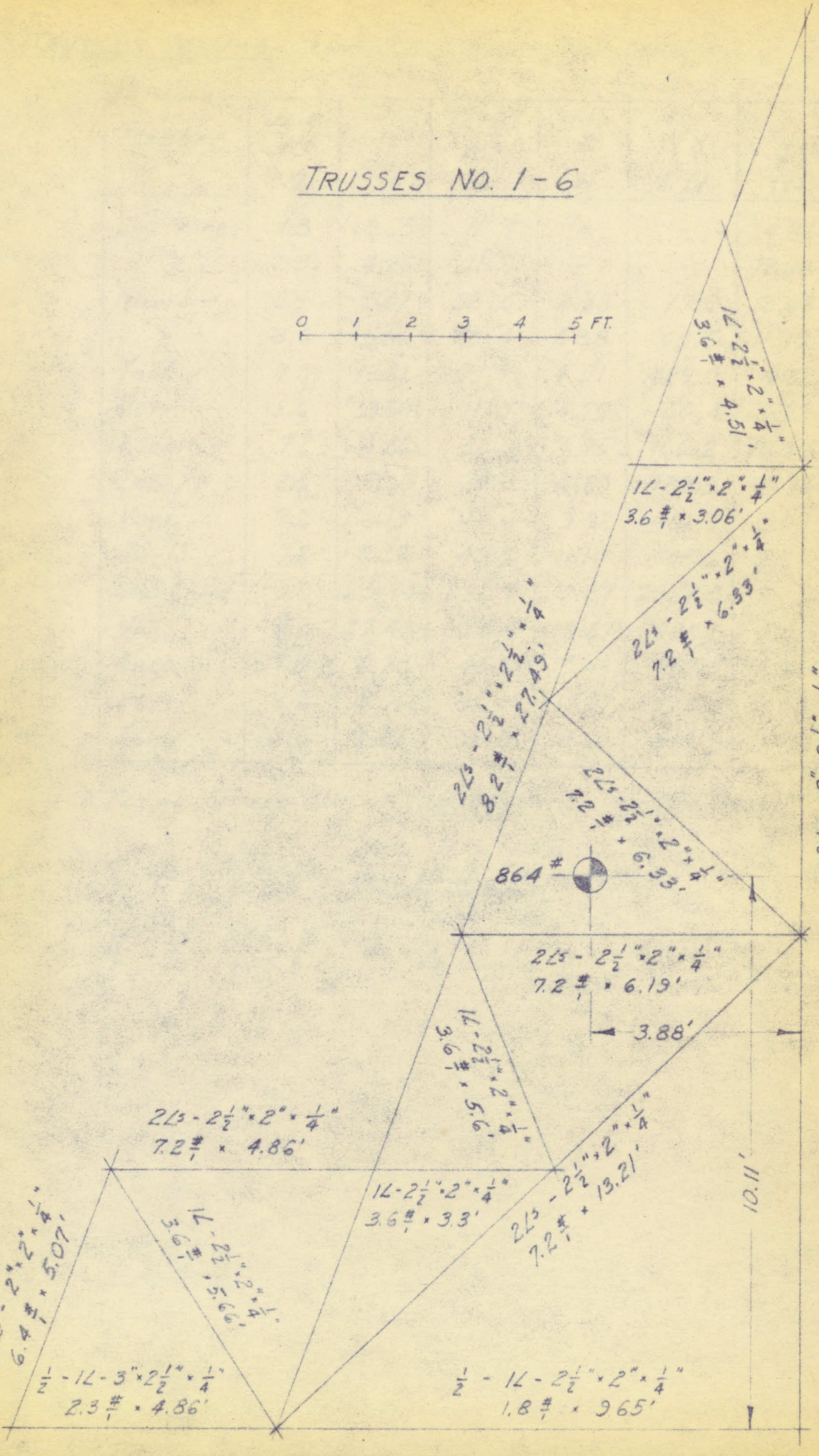
D. APPENDIX

A necessary preliminary step to the weight distribution was the calculation of truss weight and center of gravity for each different bent. Also, the unit weights for roofs, walls and other components were required. All of these elementary calculations are appended here to keep from confusing the main steps of this thesis.

1. TRUSS WEIGHTS

The various trusses have been diagramed and the tabular calculations of centers of gravity are self-explanatory.

TRUSSES NO. 1-6



TRUSSES No. 1-6 cont'd

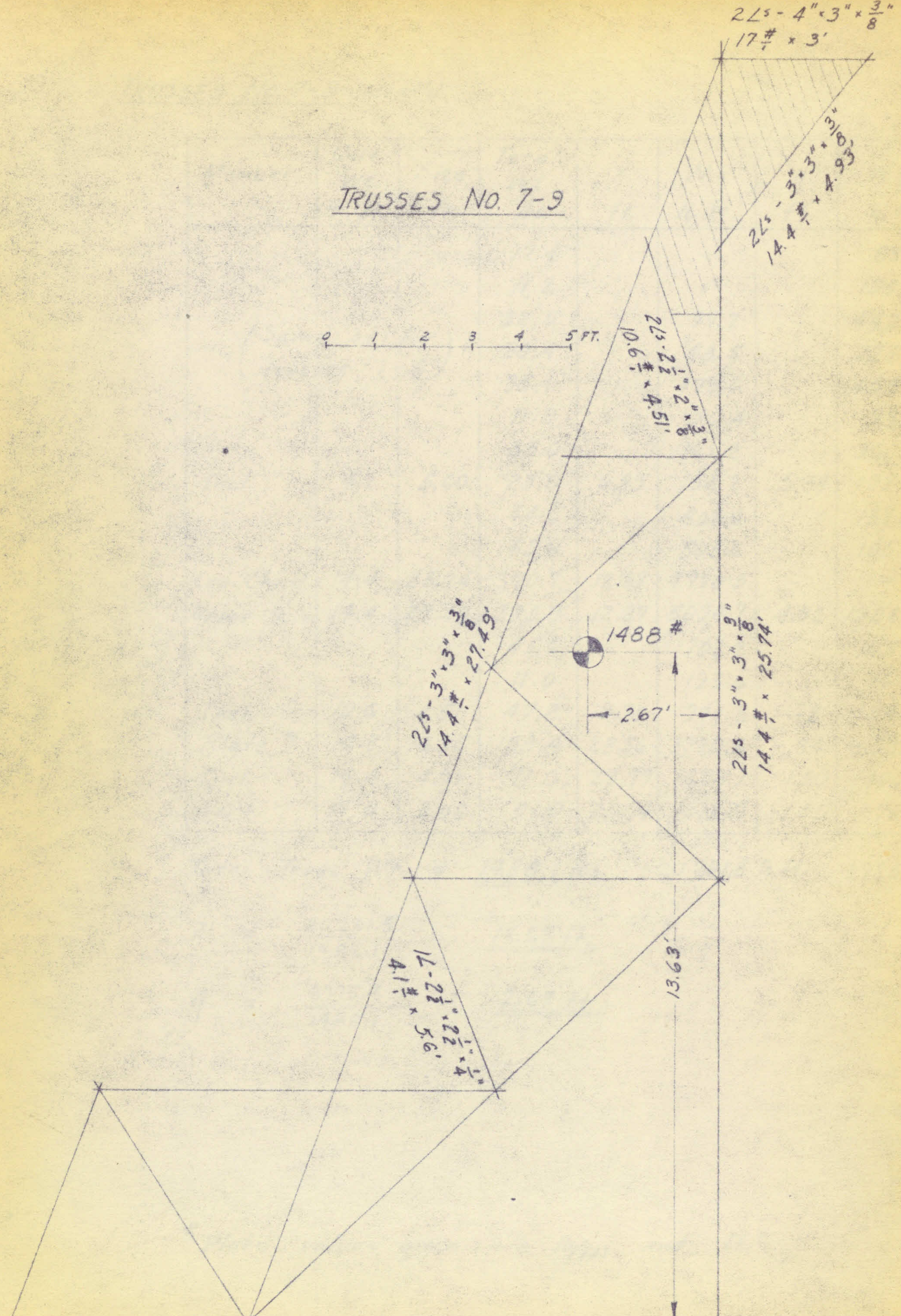
Member	Unit Wt. lb./ft.	Size ft.	Total Wt. lbs.	x ft.	Wx lb. ft.	y ft.	Wy lb. ft.
Ctr. Vert.	1.8	9.65	17.4	–	–	4.82	83.7
" "	2.3	4.86	11.2	–	–	12.08	135.0
Monitor	6.4	5.07	32.5	2.38	77.2	13.63	442.3
"	3.6	5.66	20.4	2.38	48.5	11.19	228.0
Incl.	7.2	13.21	95.1	4.51	429.0	4.82	458.4
Vert.	3.6	3.30	11.9	4.75	56.4	6.22	73.9
Monitor	7.2	4.86	35.0	4.75	166.2	10.30	360.4
Incl.	3.6	5.60	20.2	6.89	138.9	5.34	107.7
Vert.	7.2	6.19	44.6	9.02	402.0	3.09	137.7
Incl.	7.2	6.33	45.6	11.16	508.6	2.31	105.3
Low. Chord	9.0	25.74	231.7	12.87	2981.5	–	–
Up. "	8.2	27.49	225.4	12.87	2701.1	4.82	1086.5
Incl.	7.2	6.33	45.6	15.43	703.2	2.31	105.3
Vert.	3.6	3.06	11.0	17.57	193.6	1.53	16.9
Incl.	3.6	4.51	16.2	19.70	319.9	.72	11.7

Total Truss Wt. = 863.8 lbs. 8726.1 lb. ft 3352.8 lb. ft.

$$\bar{x} = \frac{8726.1}{863.8} = \underline{\underline{10.11 \text{ ft.}}}$$

$$\bar{y} = \frac{3352.8}{863.8} = \underline{\underline{3.88 \text{ ft.}}}$$

TRUSSES NO. 7-9



TRUSSES No. 7-9 cont'd

Member	Unit Wt. lb./ft.	Size ft.	Total Wt. lbs.	x ft.	Wx lb. ft.	y ft.	Wy lb. ft.
(Same as for trusses 1-6)			17.4		-		83.7
			11.2		-		135.0
			32.5		77.2		442.3
			20.4		48.5		228.0
			95.1		429.0		458.4
			11.9		56.4		73.9
			35.0		166.2		360.4
Incl.	4.1	5.60	23.0	6.89	158.2	5.34	122.6
			44.6		402.0		137.7
			45.6		508.6		105.3
Low. Chord	14.4	25.74	370.7	12.87	4770.3	-	-
Upper "	14.4	27.49	395.9	12.87	5094.7	4.82	1908.0
			45.6		703.2		105.3
			11.0		193.6		16.9
Incl.	10.6	4.51	47.8	19.70	941.8	.72	34.4
Plate *	15.3	10.35	158.4	23.56	3730.2	-.34	-53.6
Incl.	14.4	4.93	71.0	23.79	1688.9	-1.50	-106.5
Vert.	17.0	3.00	51.0	25.74	1312.7	-1.50	-76.5

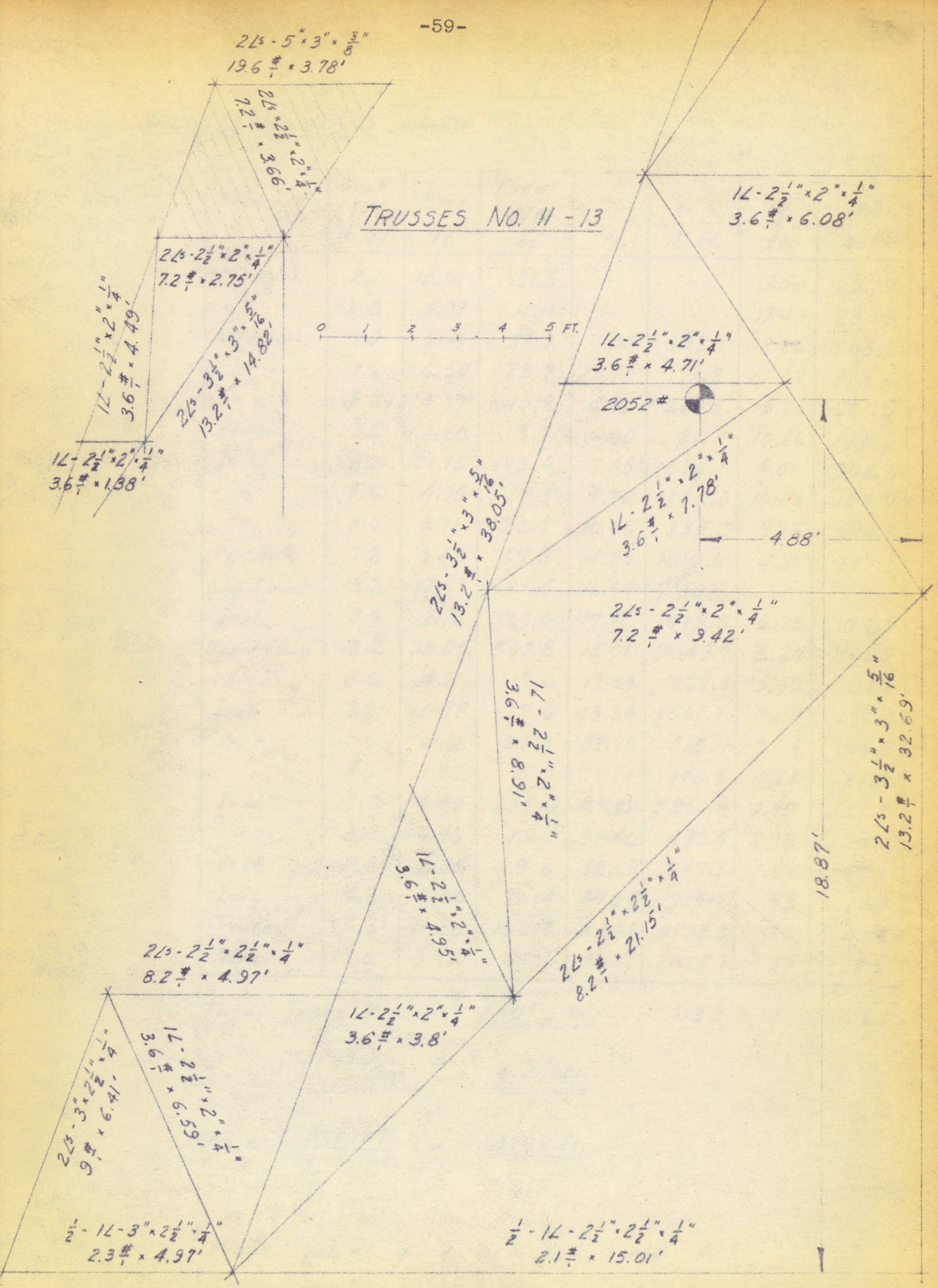
Total Truss Wt. = 1488.1 lbs.   20,281.5 lb.ft.   3975.3 lb.ft.

$$\bar{x} = \frac{20,281.5}{1488.1} = \underline{\underline{13.63 \text{ ft.}}}$$

$$\bar{y} = \frac{3,975.3}{1488.1} = \underline{\underline{2.67 \text{ ft.}}}$$

*Plate values given are lb/ft² and ft.²

TRUSSES NO. 11 - 13



TRUSSES No. 11-13 cont'd

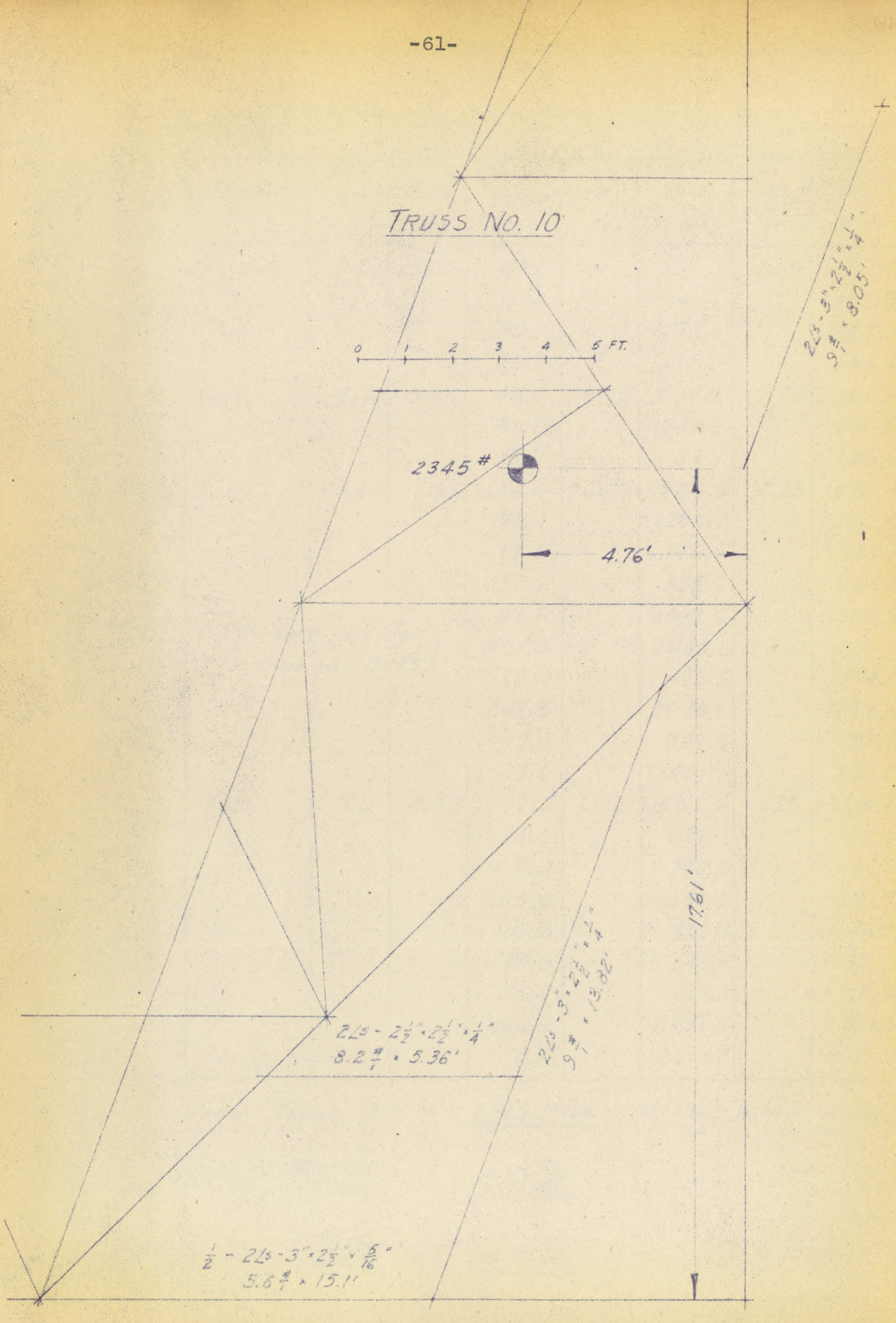
Member	Unit Wt. lb./ft.	Size ft.	Total Wt. lbs.	x ft.	Wx lb. ft.	y ft.	Wy lb. ft.
Ctr. Vert.	2.1	15.01	31.5	-	-	7.50	236.7
" "	2.3	4.97	11.4	-	-	17.49	199.9
Monitor	9.0	6.41	57.7	3.00	173.1	18.86	1088.0
"	3.6	6.59	23.7	3.00	71.2	16.37	388.4
"	8.2	4.97	40.8	6.00	244.5	15.25	621.5
Vert.	3.6	3.80	13.7	6.00	82.1	10.86	148.6
Incl.	8.2	21.15	173.4	7.45	1292.1	7.51	1302.5
"	3.6	4.95	17.8	8.22	146.5	10.03	178.7
"	3.6	8.91	32.1	10.45	335.2	9.19	294.8
Vert.	7.2	9.42	67.8	14.90	1010.6	4.71	319.5
Low. Chord	13.2	32.69	431.5	16.34	7050.8	-	-
Incl.	3.6	7.78	28.0	17.12	479.5	6.23	174.5
Upper Chord	13.2	38.05	502.3	18.01	9045.7	8.26	4148.7
Vert.	3.6	4.71	17.0	19.34	327.9	5.40	91.6
Incl.	7.2	10.78	77.6	19.34	1501.1	3.04	236.0
Vert.	3.6	6.08	21.9	23.79	520.7	3.04	66.5
"	3.6	1.38	5.0	28.24	140.3	4.23	21.0
Incl.	13.2	14.82	195.6	29.91	5851.4	1.90	251.7
"	3.6	4.49	16.2	30.46	492.4	2.90	46.9
Vert.	7.2	2.75	19.8	32.69	647.3	1.37	27.1
Incl.	7.2	3.66	26.4	34.35	905.2	.75	19.8
Plate *	15.3	10.88	166.5	34.44	5733.5	.30	49.4
Vert.	19.6	3.78	74.1	36.02	2668.7	-.39	-28.9

Total Truss Wt. = 2,051.6 lbs. 38,719.8 lb. ft. 10,002.7 lb. ft.

$$\bar{x} = \frac{38,719.8}{2,051.6} = \underline{\underline{18.87 \text{ ft.}}}$$

$$\bar{y} = \frac{10,002.7}{2,051.6} = \underline{\underline{4.88 \text{ ft.}}}$$

*Plate values given are lb/ft² and ft.²


TRUSS NO. 10
0
1
2
3
4
5 FT.
2345#
4.76'
17.61'
2∠s - 3" × 2½" × ¼"
9#/' × 8.05'
2∠s - 3" × 2½" × ¼"
9#/' × 13.32'
2∠s - 2½" × 2½" × ¼"
8.2#/' × 5.36'
½ - 2∠s - 3" × 2½" × 5/16"
5.6#/' × 15.1'

TRUSS No. 10. cont'd

Member	Unit Wt. lb./ft.	Size ft.	Total Wt. lbs.	x ft.	Wx lb. ft.	y ft.	Wy lb. ft.
Ctr. Vert.	5.6	15.01	84.1	–	–	7.50	630.4
			11.4		–		199.9
			57.7		173.1		1088.0
			23.7		71.2		388.4
Vert.	8.2	5.36	44.0	4.75	208.8	7.55	331.8
			40.8		244.5		621.5
			13.7		82.1		148.6
Incl.	9.0	13.82	124.4	6.47	804.7	4.23	526.1
			173.4		1292.1		1302.5
			17.8		146.5		178.7
			32.1		335.2		294.8
			67.8		1010.6		319.5
(Same as for trusses 11-13)			431.5		7050.8		–
			28.0		479.5		174.5
			502.3		9045.7		4148.7
			17.0		327.9		91.6
			77.6		1501.1		236.0
Incl.	9.0	8.05	72.5	21.57	1562.8	-1.37	- 99.3
			21.9		520.7		66.5
			5.0		140.3		21.0
			195.6		5851.4		371.7
			16.2		492.4		46.9
			19.8		647.3		27.1
			26.4		905.2		19.8
			166.5		5733.5		49.4
			74.1		2668.7		-28.9

Total Truss Wt. = 2345.3 lbs. 41,296.1 lb. ft. 11,155.2 lb. ft.

$$\bar{x} = \frac{41{,}296.1}{2{,}345.3} = 17.61 \text{ ft.}$$

$$\bar{y} = \frac{11{,}155.2}{2{,}345.3} = 4.76 \text{ ft.}$$