





THESIS

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A Structural Analysis of The  
Michigan State College Auditorium

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

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Figure 1: Schematic representation of the experimental design. The diagram shows a sequence of events: 'Stimulus' (a box with a question mark), 'Response' (a box with a question mark), 'Feedback' (a box with a question mark), and 'Outcome' (a box with a question mark). Arrows indicate the flow from Stimulus to Response, Response to Feedback, and Feedback to Outcome. A feedback loop arrow connects Outcome back to Stimulus.

1. The first of these is the fact that the United States has a long and distinguished record of leadership in the field of human rights. This record is reflected in the many treaties and conventions to which the United States has adhered, and in the many efforts it has made to promote human rights throughout the world.

1. The following conditions shall apply to the use of the information, data, and materials provided by the Government to the contractor:

On 10/10/1964, the following information was received from the Bureau of the Federal Bureau of Investigation, Washington, D.C., regarding the above captioned matter:

[illegible]

1. The first step in the process of identifying a problem is to recognize that a problem exists. This involves gathering information about the situation and identifying the specific issue that needs to be addressed.

[illegible][illegible]

## ANALYSIS OF ROOF LOADS

The roof is a gable type with a pitch of 12 in 12. The roof is covered with 1/2 in. thick asphalt shingles. The roof is supported by a system of trusses and rafters. The trusses are spaced 20 ft. apart and the rafters are spaced 16 in. apart. The roof is supported by a system of columns and beams. The columns are spaced 20 ft. apart and the beams are spaced 16 in. apart. The roof is supported by a system of walls and foundations. The walls are spaced 20 ft. apart and the foundations are spaced 16 in. apart.

## Roof Loads

The dead load for the roof is 15 lb/sq ft. The live load for the roof is 20 lb/sq ft.

The roof is supported by a system of trusses and rafters. The trusses are spaced 20 ft. apart and the rafters are spaced 16 in. apart. The roof is supported by a system of columns and beams. The columns are spaced 20 ft. apart and the beams are spaced 16 in. apart. The roof is supported by a system of walls and foundations. The walls are spaced 20 ft. apart and the foundations are spaced 16 in. apart.

The main trusses are 20 ft. long. The dead load of each truss is  $15 \times 20 = 300$  lb. The weight per foot is  $\frac{300}{20} = 15$  lb/ft.

The main rafters consist of 12 - 12 in. x 12 in. x 12 ft. The dead load is  $15 \times 12 = 180$  lb.

The total dead load is as follows:

|                                    |         |
|------------------------------------|---------|
| 12 in. x 12 in. x 12 ft. = 180 lb. |         |
| 20 ft. x 15 lb/ft. = 300 lb.       | 580 lb. |
| 4 in. x 8 in. x 12 ft. = 48 lb.    |         |
| 20 ft. x 15 lb/ft. = 300 lb.       | 448 lb. |
| 8 in. x 8 in. x 12 ft. = 96 lb.    |         |
| 20 ft. x 15 lb/ft. = 300 lb.       | 396 lb. |
| 4 in. x 4 in. x 12 ft. = 48 lb.    |         |
| 20 ft. x 15 lb/ft. = 300 lb.       | 348 lb. |
| <u>1776 lb.</u>                    |         |

Roof Framing, etc. = 300 lb.

|                                 |         |
|---------------------------------|---------|
| 4 in. x 8 in. x 12 ft. = 48 lb. |         |
| 20 ft. x 15 lb/ft. = 300 lb.    | 348 lb. |
| 8 in. x 8 in. x 12 ft. = 96 lb. |         |
| 20 ft. x 15 lb/ft. = 300 lb.    | 396 lb. |
| 4 in. x 4 in. x 12 ft. = 48 lb. |         |
| 20 ft. x 15 lb/ft. = 300 lb.    | 348 lb. |
| <u>1776 lb.</u>                 |         |

Total = 1776 lb.

The weight per foot is  $\frac{1776}{20} = 88.8$  lb/ft.

This is distributed to the two trusses.  
For one truss  $\frac{1776}{2} = 888$  lb.

[illegible][illegible]

Upper Grand, 1911-1912, 1913-1914:

|                   |                                        |
|-------------------|----------------------------------------|
| Roof              | 5140 lbs.                              |
| Walling           | 1000                                   |
| Framing           | 500                                    |
| Floor             | 5110                                   |
| Plates and rivets | 7                                      |
|                   | <hr/> 11,057 lbs.    Total 11,057 lbs. |

The authors thank the following people for their help:

$$\frac{1,000}{10} = 100 \text{ in.}^2 / \text{ft.}^2 \text{ of } 1000 \text{ sq. ft. of } 100 \text{ in.}^2 / \text{ft.}^2$$

10 11. / 12. 13. 14. 15. 16. 17. 18. 19. 20. 21. 22. 23. 24. 25. 26. 27. 28. 29. 30. 31. 32. 33. 34. 35. 36. 37. 38. 39. 40. 41. 42. 43. 44. 45. 46. 47. 48. 49. 50. 51. 52. 53. 54. 55. 56. 57. 58. 59. 60. 61. 62. 63. 64. 65. 66. 67. 68. 69. 70. 71. 72. 73. 74. 75. 76. 77. 78. 79. 80. 81. 82. 83. 84. 85. 86. 87. 88. 89. 90. 91. 92. 93. 94. 95. 96. 97. 98. 99. 100. 101. 102. 103. 104. 105. 106. 107. 108. 109. 110. 111. 112. 113. 114. 115. 116. 117. 118. 119. 120. 121. 122. 123. 124. 125. 126. 127. 128. 129. 130. 131. 132. 133. 134. 135. 136. 137. 138. 139. 140. 141. 142. 143. 144. 145. 146. 147. 148. 149. 150. 151. 152. 153. 154. 155. 156. 157. 158. 159. 160. 161. 162. 163. 164. 165. 166. 167. 168. 169. 170. 171. 172. 173. 174. 175. 176. 177. 178. 179. 180. 181. 182. 183. 184. 185. 186. 187. 188. 189. 190. 191. 192. 193. 194. 195. 196. 197. 198. 199. 200. 201. 202. 203. 204. 205. 206. 207. 208. 209. 210. 211. 212. 213. 214. 215. 216. 217. 218. 219. 220. 221. 222. 223. 224. 225. 226. 227. 228. 229. 230. 231. 232. 233. 234. 235. 236. 237. 238. 239. 240. 241. 242. 243. 244. 245. 246. 247. 248. 249. 250. 251. 252. 253. 254. 255. 256. 257. 258. 259. 260. 261. 262. 263. 264. 265. 266. 267. 268. 269. 270. 271. 272. 273. 274. 275. 276. 277. 278. 279. 280. 281. 282. 283. 284. 285. 286. 287. 288. 289. 290. 291. 292. 293. 294. 295. 296. 297. 298. 299. 300. 301. 302. 303. 304. 305. 306. 307. 308. 309. 310. 311. 312. 313. 314. 315. 316. 317. 318. 319. 320. 321. 322. 323. 324. 325. 326. 327. 328. 329. 330. 331. 332. 333. 334. 335. 336. 337. 338. 339. 340. 341. 342. 343. 344. 345. 346. 347. 348. 349. 350. 351. 352. 353. 354. 355. 356. 357. 358. 359. 360. 361. 362. 363. 364. 365. 366. 367. 368. 369. 370. 371. 372. 373. 374. 375. 376. 377. 378. 379. 380. 381. 382. 383. 384. 385. 386. 387. 388. 389. 390. 391. 392. 393. 394. 395. 396. 397. 398. 399. 400. 401. 402. 403. 404. 405. 406. 407. 408. 409. 410. 411. 412. 413. 414. 415. 416. 417. 418. 419. 420. 421. 422. 423. 424. 425. 426. 427. 428. 429. 430. 431. 432. 433. 434. 435. 436. 437. 438. 439. 440. 441. 442. 443. 444. 445. 446. 447. 448. 449. 450. 451. 452. 453. 454. 455. 456. 457. 458. 459. 460. 461. 462. 463. 464. 465. 466. 467. 468. 469. 470. 471. 472. 473. 474. 475. 476. 477. 478. 479. 480. 481. 482. 483. 484. 485. 486. 487. 488. 489. 490. 491. 492. 493. 494. 495. 496. 497. 498. 499. 500. 501. 502. 503. 504. 505. 506. 507. 508. 509. 510. 511. 512. 513. 514. 515. 516. 517. 518. 519. 520. 521. 522. 523. 524. 525. 526. 527. 528. 529. 530. 531. 532. 533. 534. 535. 536. 537. 538. 539. 540. 541. 542. 543. 544. 545. 546. 547. 548. 549. 550. 551. 552. 553. 554. 555. 556. 557. 558. 559. 560. 561. 562. 563. 564. 565. 566. 567. 568. 569. 570. 571. 572. 573. 574. 575. 576. 577. 578. 579. 580. 581. 582. 583. 584. 585. 586. 587. 588. 589. 590. 591. 592. 593. 594. 595. 596. 597. 598. 599. 600. 601. 602. 603. 604. 605. 606. 607. 608. 609. 610. 611. 612. 613. 614. 615. 616. 617. 618. 619. 620. 621. 622. 623. 624. 625. 626. 627. 628. 629. 630. 631. 632. 633. 634. 635. 636. 637. 638. 639. 640. 641. 642. 643. 644. 645. 646. 647. 648. 649. 650. 651. 652. 653. 654. 655. 656. 657. 658. 659. 660. 661. 662. 663. 664. 665. 666. 667. 668. 669. 670. 671. 672. 673. 674. 675. 676. 677. 678. 679. 680. 681. 682. 683. 684. 685. 686. 687. 688. 689. 690. 691. 692. 693. 694. 695. 696. 697. 698. 699. 700. 701. 702. 703. 704. 705. 706. 707. 708. 709. 710. 711. 712. 713. 714. 715. 716. 717. 718. 719. 720. 721. 722. 723. 724. 725. 726. 727. 728. 729. 730. 731. 732. 733. 734. 735. 736. 737. 738. 739. 740. 741. 742. 743. 744. 745. 746. 747. 748. 749. 750. 751. 752. 753. 754. 755. 756. 757. 758. 759. 760. 761. 762. 763. 764. 765. 766. 767. 768. 769. 770. 771. 772. 773. 774. 775. 776. 777. 778. 779. 780. 781. 782. 783. 784. 785. 786. 787. 788. 789. 790. 791. 792. 793. 794. 795. 796. 797. 798. 799. 800. 801. 802. 803. 804. 805. 806. 807. 808. 809. 810. 811. 812. 813. 814. 815. 816. 817. 818. 819. 820. 821. 822. 823. 824. 825. 826. 827. 828. 829. 830. 831. 832. 833. 834. 835. 836. 837. 838. 839. 840. 841. 842. 843. 844. 845. 846.

10/10/1961 - 10/10/1961

Top of the Hill, March 10.

[illegible]

LOWER ANDERSON 1 TO 3, LENSES 10:

The total weight is two pounds eight ounces plus one ounce plus one ounce, which equals four pounds.

The "Lilac" lot is to run to 11,100 lbs., and the  
"Blue" lot, 11,100 lbs., and the "Red" lot, 11,100 lbs.  
This 30,100 lbs. of wool is for the 1st and 2nd.

For time: 11 (initial) hrs. by 11 students and 11 hrs.  
of students. The same amount of time was used for the other 11 hrs.  
11 x 11 = 121 hrs./11 hrs.  
11 x 11 = 121 hrs./11 hrs.  
11 x 11 = 121 hrs./11 hrs.  
11 x 11 = 121 hrs./11 hrs.

[illegible]

| Bar          |              | Truss T 2 (One-half)          |                         |             |
|--------------|--------------|-------------------------------|-------------------------|-------------|
|              | 2-15 in.     | s @ 40 lbs.                   | - 46.5 ft. long         | 3720.       |
|              | "            | "                             | 29.5 ft.                | 2360.       |
|              | 1-Tee-18 in. | x 79 ft.                      | @ 97 lbs.               | 7660.       |
| 1-0          | 2            | -6 in. x 4 in. x 3/8 in.      | x 9 1/2 ft. @ 12.3 lbs. | 243.        |
| 2-1          | 2            | -6 x 6 x 5/8 x 15 ft.         | @ 24.2 lbs.             | 727.        |
| 3-2          | 2            | -6 x 4 x 3/8 x 10 1/2 ft.     | @ 12.3 lbs.             | 258.        |
| 4-3          | 2            | -6 x 6 x 3/8 x 16 ft.         | @ 14.9                  | 477.        |
| 5-4          | 2            | -5 x 3 x 3/8 x 11 3/4 ft.     | @ 9.8 lbs.              | 231.        |
| 6-5          | 2            | -4 x 4 x 3/8 x 16 1/2 ft.     | @ 9.8 lbs.              | 324.        |
| 7-6          | 2            | -5 x 3 x 3/8 x 13 ft.         | @ 9.8 lbs.              | 255.        |
| 8-7          | 2            | -6 x 6 x 3/8 x 23 3/4         | @ 14.9 lbs.             | 707.        |
| 9-8          | 2            | -6 x 6 x 3/8 x 18 1/2         | @ 14.9 lbs.             | 552.        |
| 10-9         | 2            | -6 x 4 x 7/16 x 28 ft.        | @ 14.3 lbs.             | 800.        |
| 10-10        | 1            | -3 1/2 x 3 1/2 x 3/8 x 24 1/2 | @ 8.5 lbs.              | 208.        |
|              |              |                               |                         | <hr/> 18522 |
| Total Weight |              |                               |                         | 37,044 lbs. |
| Wt./panel    |              |                               |                         | 3,710 lbs.  |

|              |          | Truss T 3 (One-half)          |                      |             |
|--------------|----------|-------------------------------|----------------------|-------------|
|              | 2-18 in. | s                             | - 46.5 ft. @ 58 lbs. | 5400.       |
|              | "        | "                             | 29.5 ft. @ 58 lbs.   | 3425.       |
|              | 1-18 in. | Tee x 79 ft.                  | @ 150 lbs.           | 11850.      |
| 1-A          | 2        | -6 x 6 x 11/16 x 9 1/2        | @ 26.5 lbs.          | 504.        |
| 2-1          | 2        | -8 x 8 x 5/8 x 15 ft.         | @ 32.7 lbs.          | 981.        |
| 3-2          | 2        | -6 x 6 x 9/16 x 10 1/2        | @ 21.9 lbs.          | 460.        |
| 4-3          | 2        | -6 x 6 x 11/16 x 16           | @ 26.5 lbs.          | 848.        |
| 5-4          | 2        | -6 x 6 x 7/16 x 11 3/4        | @ 17.2 lbs.          | 402.        |
| 6-5          | 2        | -6 x 6 x 7/16 x 16 1/2        | @ 17.2 lbs.          | 564.        |
| 7-6          | 2        | -5 x 3 x 3/8 x 13             | @ 9.8 lbs.           | 254.        |
| 8-7          | 2        | -6 x 6 x 9/16 x 23 3/4        | @ 21.9 lbs.          | 1040.       |
| 9-8          | 2        | -8 x 6 x 7/16 x 18 1/2        | @ 20.2 lbs.          | 747.        |
| 10-9         | 2        | -6 x 6 x 5/8 x 28             | @ 24.2 lbs.          | 1357.       |
| 10-10        | 1        | -3 1/2 x 3 1/2 x 3/8 x 24 1/2 | @ 8.5 lbs.           | 208.        |
|              |          |                               |                      | <hr/> 28040 |
| Total Weight |          |                               |                      | 56,080 lbs. |
| Wt./ft.      |          |                               |                      | 372 lbs.    |
| Wt./panel    |          |                               |                      | 5,610 lbs.  |





$$\frac{2930}{10} = 293 \text{ lbs./panel. Say } 300 \text{ lbs.}$$

## Purlins:

$$14-12 \text{ in WF} - 20 \text{ ft. @ } 25 \text{ lbs.} = 7000 \text{ lbs.}$$

$$\text{WF or } 700 \text{ lbs./panel}$$

## Truss T 1 (One-half)

|                |                                                                      |                   |
|----------------|----------------------------------------------------------------------|-------------------|
| Top Chord 2    | -6 x 4 x $\frac{1}{2}$ x $35\frac{3}{4}$ @ 16.2 lbs.                 | 1156.             |
| Bottom Chord " | -5 x $3\frac{1}{2}$ x p x $35\frac{3}{4}$ ft. @ 13.6                 | 972.              |
| 1-A 2          | - $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{2}$ x 9 ft. @ 9.4      | 169.              |
| 2-1 2          | - $3\frac{1}{2}$ x 3 x $\frac{3}{8}$ x $8\frac{3}{4}$ @ 7.9 lbs.     | 138.              |
| 3-2 2          | - $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{3}{8}$ x 8 ft. @ 7.2 lbs. | 115.              |
| 4-3 2          | - $3\frac{1}{2}$ x 3 x $\frac{5}{16}$ x 8 ft. @ 6.6 lbs.             | 106.              |
| 5-4 2          | - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$ x 7 ft. @ 4.1 lbs. | 58.               |
| 6-5 2          | -4 x 3 x $\frac{5}{16}$ x $11\frac{3}{4}$ @ 7.2 lbs.                 | 169.              |
| 7-6 2          | -3 x $2\frac{1}{2}$ x $\frac{1}{4}$ x $7\frac{1}{2}$ @ 4.5 lbs.      | 68.               |
| 8-7 2          | -4 x 3 x $\frac{5}{16}$ x 12 @ 7.2 lbs.                              | 173.              |
| 9-8 2          | -3 x $2\frac{1}{2}$ x $\frac{1}{4}$ x $8\frac{3}{4}$ @ 4.5 lbs.      | 79.               |
| 10-9 2         | -3 x $2\frac{1}{2}$ x $\frac{5}{16}$ x $13\frac{1}{4}$ @ 10.5 lbs.   | 278.              |
| 10-10 1        | - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$ x 11 @ 3.19 lbs.   | 34.               |
|                |                                                                      | <u>3,515 lbs.</u> |
|                | Wt./panel 352 lbs x 2 = 704 lbs.                                     |                   |

## Upper Chord panel load, T 1:

|              |                                       |
|--------------|---------------------------------------|
| Roof         | 2160                                  |
| Sway framing | 300                                   |
| Purlins      | 700                                   |
| Truss        | 704                                   |
| Total        | <u>3864 lbs./panel. Say 3870 lbs.</u> |

## The lower chord load is as follows:

|                  |                        |
|------------------|------------------------|
| Framing          | 4 lbs./sq. ft.         |
| Lath and plaster | 10 lbs./sq. ft.        |
| Mechanical etc.  | 4                      |
| Total            | <u>18 lbs./sq. ft.</u> |

$$18 \times 20 \times 6.7 = 2400 \text{ lbs./panel}$$

The stresses in each member caused by the above dead loads are solved graphically and the results itemized in tables as shown.

## II Snow Loads

For the territory in which the building is situated, a maximum snow load of around 15 lbs./sq. ft. for the horizontal projection is recommended. A minimum of 5 lbs./sq. ft was chosen. 10 lbs./sq. ft. was chosen as an index value and the maximum and minimum values found by proportion. The panel load for T 2 and T 3 =  $10 \times 25 \times 15 = 3750$  lbs. The panel load for T 1  $10 \times 20 \times 6\frac{2}{3} = 1340$  lbs. The stresses caused by snow and ice were found graphically and the maximum and minimum effects recorded.

### III Wind Load

The normal pressure on a roof is given by Duchemin's formula:

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A}$$

where  $P$  = intensity of pressure on a vertical surface normal to the wind,  $P_n$  = intensity of normal pressure on a sloping surface and  $A$  = angle of the surface with the horizontal. Maximum  $P$  is taken equal to 30 lbs./sq. ft. Minimum  $P$  = one-third of maximum or 10 lbs./sq. ft.

$$P_n = 30 \frac{(.353)}{1.125} = 18.75 \text{ lbs./sq. ft.}$$

Say 19 lbs./sq. ft. Maximum

For T 2 and T 3, the panel load normal to roof =  
 $19 \times 25 \times 15.7 = 7500 \text{ lbs.}$

For T 1:

$$P_n = 30 \frac{2(.366)}{1.134} = 19.35 \text{ lbs./sq. ft.}$$

Say 20 lbs./sq. ft. Maximum

Panel load normal to roof =  $20 \times 20 \times 7.5 = 3000 \text{ lbs.}$

The stresses in all bars of each truss were also found graphically. To find the end reactions, it was assumed that the horizontal components at each end were equal.

The maximum total stress in each bar was found by the following formulas:  $S = S \text{ dead} + S \text{ wind (Max.)} + S \text{ snow (Min.)}$   
 $S = S \text{ dead} + S \text{ wind (Min.)} + S \text{ snow (Max.)}$

The largest value of  $S$  is tabulated. The unit stresses in each member are also tabulated. For all lower chord members, four rivet holes are deducted and all other tension members, two holes are deducted.

The allowable stresses are taken from A.I.S.C. specifications. Allowable stress in tension = 20,000 lbs/sq. in. For compression, allowable stress =  $17,000 - 0.485 \frac{l}{r}$  for axially loaded columns with  $\frac{l}{r}$  less than 120 and  $\frac{18000}{1 + \frac{l^2}{18000r^2}}$  for values of  $\frac{l}{r}$  greater than 120.  $l$  = unbraced length of column and  $r$  is the corresponding radius of gyration.

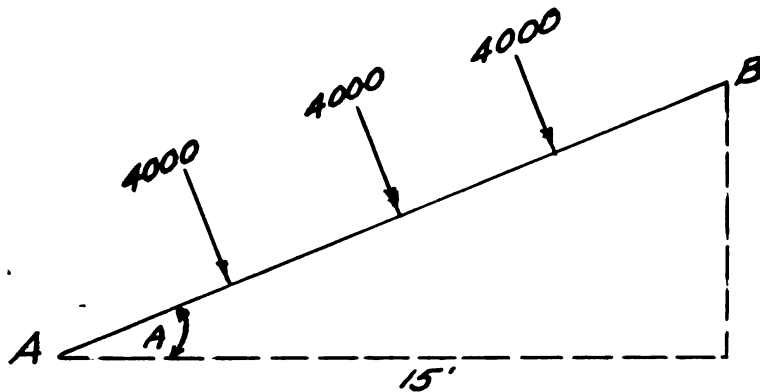
In addition to the direct stresses, the purlins, and upper chord members must be investigated for bending stresses. The upper chord members carry the weight of the uniformly distributed dead, wind, and snow.



loads. These are transmitted to the chords by the purlins.

The purlins for T 2 and T 3 are identical. They are 25 ft. long; and spaced 4 ft. on centers. Each one supports a uniform roof slab, wind and snow loads, and its own weight. The uniform load  $= 4 \times 19 + 25 \cos A + 4(5 + 14) (\cos A) = 160 \text{ lbs./ft.}$   
 $M_{\text{max}} = \frac{160 \times (25)^2}{8} = 12,500 \text{ ft. lbs.}$

$$S = \frac{M_{\text{c}}}{I} = \frac{12,500 \times 12 \times 3.95}{185.4} = 4,800 \text{ lbs./sq. in. Safe.}$$



$$\tan A = \frac{29}{74.5}$$

$$\cos A = \frac{15}{AB}$$

$$A = 21^\circ 15'$$

$$AB = 16'$$

The purlin concentration on member AB  $= 160 \times 25 = 4000 \text{ lbs.}$   
 $M_{\text{max}} = 3000 \times 8 = 4000 \times 4 = 32,000 \text{ ft. lbs.}$

The upper chord of truss T 2 is a 12 in. - 97 lb. tee section.

$$\text{Maximum } M = 32,000 + \frac{27 \cos A \times 16.1^2}{8} = 34,920 \text{ ft. lbs.}$$

$$S = \frac{M_{\text{c}}}{I} = \frac{34,920 \times 12 \times 4.81}{904} = 2,250 \text{ m/b. Total direct stress = } 14,430 \text{ lb./sq. in. as}$$

compared to 16,500 lb./sq. in. allowable.

The upper chord of truss T 3 is a 12 in. Tee section.

$$\text{Maximum } M = 32,000 + \frac{150 \cos A (16.1)^2}{8} = 36,540 \text{ ft. lbs.}$$

$$S = \frac{36,540 \times 12 \times 4.13}{1222.7} = 1475 \text{ lb./sq. in.}$$

Total direct and bending stress  $= 14,800 + 1475 = 16,275 \text{ lb./sq. in.}$   
 compared to 16,400 lb./sq. in. allowable.

The purlins of truss T 1 are spaced 6 ft. 8 in. on centers and fastened at panel points. The loads are therefore transmitted directly to the panel points.

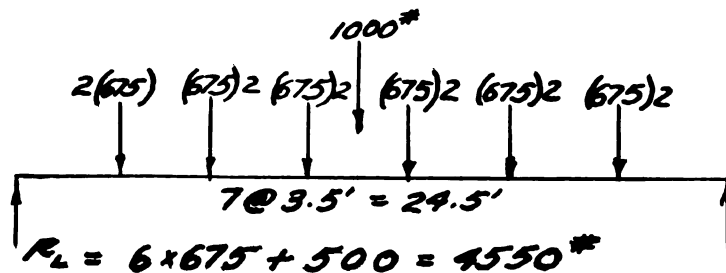


The loads acting on the purlins =  $20 \times 7.12 + (14+5) \times (6 \frac{2}{3}) + 25 = 295 \text{ lb./ft. normal to roof.}$

$$M_{\max} = \frac{295 \times (20)^2}{8} = 14,750 \text{ ft. lbs.}$$

$$S = \frac{M_{\max}}{I} = \frac{14,750 \times 12 \times 5.93}{183.4} = 5,820 \text{ lb./sq. in. p.s.i.}$$

The ceiling framing of the main auditorium consists of six 8 in x 4 in. - 10 lb. joists per panel. These are supported by 12 in. W F 25 lb. sections which in turn are connected to the lower panel points of the truss. The load transmitted to the 25 lb. W F sections are its own weight plus the weight of the joists, the lath and plaster ceiling, and a walkway. The walkway weighs 500 lbs./panel. The live load will be assumed as 500 lbs./panel making the total concentration = 1000 lbs.



The concentration of the 10 lb. sections =  $(6.5 \times 10 \times 15 + 10 \times 15) = 675 \text{ lbs. each.}$

$$M_{\max} = 4550 \times 12 - 675 (1.75 + 5.25 + 8.75) + \frac{25 \times (20)^2}{8} = 46,985 \text{ ft. lbs.}$$

$$S = \frac{M_{\max}}{I} = \frac{46,985 \times 12 \times 5.93}{183.4} = 18,200 \text{ lbs./sq. in.}$$

Allowable stress approximately 17,000 lbs./sq. in. in compression. The live load assumed probably allows the beam to be considered safe. The 8 in. x 4 in. joists most stressed are those supporting the walkway. The load =  $\frac{500 + 10 + 3.5 \times 10}{15} = 78.3 \text{ lb/ft.}$

$$M_{\max} = \frac{78.3 \times 15^2}{8} = 2,220 \text{ ft. lbs.}$$

$$S = \frac{2,220 \times 12 \times 3.95}{50.8} = 3,420 \text{ lb./sq. in.}$$

The ceiling framing on truss T 1 consists of 8 in. x 4 in. joists suspended on 12 in. x 4 in. - 14 lb. sections fastened at alternate truss panel points. The uniform load on the joists are its own weight plus the suspended ceiling.





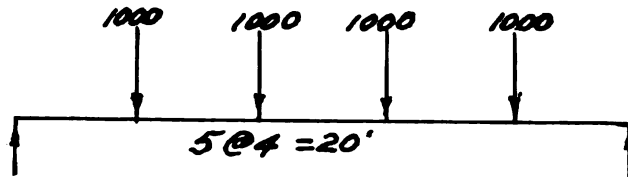
The ceiling will be taken as 15 lb./sq. ft. The joists are 4 ft. on centers. This makes the total load =  $15 \times 4 + 10 = 70$  lb./ft.

$$M_{\max} = \frac{70 \times (14\frac{1}{2})^2}{8} = 1,780 \text{ ft. lbs.}$$

$$S = \frac{1,780 \times 12 \times 3.25}{80.8} = 2,750 \text{ lb./sq. in.}$$

The end concentration due to a single joist =

$$70 \times 11.25 = 997.50 \text{ lb. Say } 1000 \text{ lbs.}$$



$$\text{Max. B. M.} = 2000 \times 10 - 1000 (6+3) + \frac{14 \times 200}{8} = 12,800 \text{ ft. lbs.}$$

$$S = \frac{12,800 \times 12 \times 3.25}{88.2} = 10,400 \text{ lb./sq. in.}$$

This concludes all of the structural frame work in connection with the trusses that is practicable to be checked for stresses.



# TRUSS T 1 WIND AND SNOW LOADS

| Member | Maximum<br>Wind | Minimum<br>Wind | Maximum<br>Snow | Minimum<br>Snow |
|--------|-----------------|-----------------|-----------------|-----------------|
| A-2    | 8,700           | 2,900           | 5,520           | 1,840           |
| B-4    | 17,700          | 5,900           | 11,100          | 3,700           |
| C-5    | 18,850          | 6,380           | 11,100          | 3,700           |
| D-7    | 25,600          | 8,350           | 13,550          | 4,520           |
| E-9    | 18,700          | 6,230           | 13,100          | 4,370           |
| F-10   | -9,850          | -3,280          | -10,200         | -3,400          |
| G-8    | -15,500         | -5,170          | -12,400         | -4,130          |
| H-6    | -20,150         | -6,720          | -13,350         | -4,450          |
| J-3    | -12,200         | -4,070          | -5,850          | -1,950          |
| K-1    | -3,200          | -1,070          | -               | -               |
| 1-2    | -10,100         | -3,370          | -6,500          | -2,160          |
| 2-3    | 10,400          | 3,470           | 6,750           | 2,250           |
| 3-4    | -8,200          | -2,730          | -6,250          | -2,080          |
| 4-5    | 3,300           | 1,100           | 1,300           | 430             |
| 5-6    | 2,600           | 870             | 4,200           | 1,400           |
| 6-7    | 1,600           | 530             | -1,150          | - 380           |
| 7-8    | -6,800          | -2,270          | - 900           | - 300           |
| 8-9    | 7,400           | 2,470           | 1,900           | 630             |
| 9-10   | -10,900         | -3,630          | -4,050          | -1,350          |
| 10-10  | -1,300          | - 430           | -1,250          | - 420           |

Note: - Indicates tension

## STRESS SUMMARY T 1

| Member | Dead Stress | Total Stress | Net Area per sq. in. | Unit Stress per sq. in. | A.L.S.C. Specifications |
|--------|-------------|--------------|----------------------|-------------------------|-------------------------|
| A-2    | 26,250      | 36,790       | 9.50                 | 3,880                   | 15,800                  |
| B-4    | 51,800      | 73,200       | 9.50                 | 7,700                   | "                       |
| C-5    | 51,800      | 74,350       | 9.50                 | 7,820                   | "                       |
| D-7    | 64,750      | 94,870       | 9.50                 | 9,980                   | "                       |
| E-9    | 61,400      | 84,470       | 9.50                 | 8,880                   | "                       |
| F-10   | -48,000     | -61,480      | 8.00                 | -7,680                  | 20,000                  |
| G-8    | -57,900     | -77,530      | 8.00                 | -9,700                  | "                       |
| H-6    | -63,750     | -88,350      | 8.00                 | -11,050                 | "                       |
| J-3    | -27,600     | -41,750      | 8.00                 | -5,220                  | "                       |
| K-1    | -           | -3,200       | 8.00                 | - 400                   | "                       |
| 1-2    | -30,750     | -43,010      | 4.60                 | -9,350                  | "                       |
| 2-3    | 29,250      | 41,900       | 4.22                 | 9,950                   | 13,200                  |
| 3-4    | -28,800     | -39,080      | 3.86                 | -10,150                 | 20,000                  |
| 4-5    | 4,000       | 7,730        | 2.58                 | 3,230                   | 10,000                  |
| 5-6    | 21,200      | 26,270       | 4.18                 | 6,260                   | 10,900                  |
| 6-7    | -8,800      | -9,950       | 2.62                 | -3,790                  | 20,000                  |
| 7-8    | -5,750      | -15,550      | 4.18                 | -3,710                  | 20,000                  |
| 8-9    | 7,500       | 15,350       | 2.62                 | 5,800                   | 11,000                  |
| 9-10   | -18,350     | -30,600      | 3.24                 | -9,450                  | 20,000                  |
| 10-10  | -8,700      | -10,420      | 2.58                 | -4,370                  | 20,000                  |



TRUSS T 2 AND T 3                      WIND AND SNOW LOADS

| Member | Maximum<br>Wind | Minimum<br>Wind | Maximum<br>Snow | Minimum<br>Snow |
|--------|-----------------|-----------------|-----------------|-----------------|
| A-2    | 55,000          | 11,700          | 37,950          | 12,650          |
| B-4    | 54,000          | 18,000          | 62,250          | 20,750          |
| C-6    | 59,800          | 19,950          | 74,700          | 24,900          |
| D-7    | 62,600          | 20,860          | 74,700          | 24,900          |
| E-9    | 45,000          | 15,000          | 51,900          | 19,300          |
| F-10   | -21,200         | -21,200         | -42,750         | -14,250         |
| G-8    | -36,400         | -12,130         | -54,000         | -18,000         |
| H-5    | -55,800         | -13,600         | -60,750         | -20,250         |
| J-3    | -40,000         | -13,330         | -37,050         | -12,320         |
| K-1    | -7,200          | -2,400          | -               | -               |
| L-1    | -32,600         | -10,870         | -42,200         | -14,070         |
| 2-3    | 19,500          | 6,500           | 22,500          | 7,500           |
| 3-4    | -15,800         | -5,270          | -24,000         | -8,000          |
| 4-5    | 10,100          | 3,370           | 15,000          | 5,000           |
| 5-6    | -2,500          | - 830           | -12,750         | -4,250          |
| 6-7    | 8,000           | 2,670           | 5,550           | 1,850           |
| 7-8    | -30,500         | -10,170         | -24,600         | -8,200          |
| 8-9    | 24,000          | 8,000           | 18,300          | 6,100           |
| 9-10   | 23,500          | -9,500          | -21,600         | -7,200          |
| 10-10  | -               | -               | -               | -               |

## TRUSS T 2

## STRESS SUMMARY

| Member | Dead Stress | Total Stress | Net Area<br>per sq. in. | Unit Stress<br>per sq. in. | A.L.S.C.<br>Specifications |
|--------|-------------|--------------|-------------------------|----------------------------|----------------------------|
| A-2    | 129,500     | 179,150      | 23.56                   | 6,280                      | 16,500                     |
| B-4    | 208,200     | 288,450      | "                       | 10,100                     | "                          |
| C-6    | 247,000     | 341,650      | "                       | 11,900                     | "                          |
| D-7    | 247,000     | 342,560      | "                       | 12,000                     | "                          |
| E-9    | 192,500     | 265,400      | "                       | 9,300                      | "                          |
| F-10   | -142,100    | -206,050     | 19.76                   | 10,450                     | 20,000                     |
| G-8    | -179,500    | -245,650     | "                       | 12,450                     | "                          |
| H-5    | -203,000    | -282,350     | "                       | 14,300                     | "                          |
| J-3    | -126,000    | -173,320     | "                       | 9,030                      | "                          |
| K-1    | -           | - 7,200      | "                       | 364                        | "                          |
| 2-1    | -126,000    | -179,070     | 12.03                   | 14,850                     | "                          |
| 2-3    | 68,200      | 97,200       | 7.22                    | 13,450                     | 14,800                     |
| 3-4    | -78,000     | -107,270     | 7.22                    | 14,880                     | 20,000                     |
| 4-5    | 44,000      | 62,570       | 5.72                    | 10,900                     | 13,200                     |
| 5-6    | -39,500     | -53,080      | 4.22                    | 12,600                     | 20,000                     |
| 6-7    | 11,500      | 21,350       | 5.72                    | 3,730                      | 12,500                     |
| 7-8    | -79,500     | -118,200     | 7.72                    | 15,450                     | 20,000                     |
| 8-9    | 54,500      | 84,600       | 8.72                    | 9,700                      | 10,200                     |
| 9-10   | -71,000     | -106,700     | 10.79                   | 9,900                      | 20,000                     |
| 10-10  | -6,000      | -6,000       | 3.46                    | 1,740                      | 20,000                     |





## TRUSS T 3      STRESS SUMMARY

| Member | Dead Stress | Total Max. Stress | Net Area Sq. in. | Unit Stress lbs. sq. in. | A.I.S.C. Specifications |
|--------|-------------|-------------------|------------------|--------------------------|-------------------------|
| A-2    | 255,300     | 305,150           | 44.09            | 6,950                    | 16,400                  |
| B-4    | 456,400     | 516,650           | "                | 11.720                   | "                       |
| C-6    | 556,500     | 651,150           | "                | 14,750                   | "                       |
| D-7    | 556,500     | 652,060           | "                | 14.800                   | "                       |
| E-7    | 454,000     | 528,900           | "                | 11,950                   | "                       |
| F-10   | -331,500    | -395,400          | 23.36            | 13,950                   | 20,000                  |
| G-8    | -422,000    | -488,150          | "                | 17,250                   | "                       |
| H-5    | -424,500    | -503,850          | "                | 17.800                   | "                       |
| J-3    | -249,000    | -301,320          | "                | 10,650                   | "                       |
| K-1    | -           | -7,200            | "                | 254                      | "                       |
| 1-2    | -249,000    | -302,000          | 16.72            | 18,050                   | "                       |
| 2-3    | 145,000     | 172,000           | 12.86            | 13,400                   | 14,800                  |
| 3-4    | -179,000    | -208,270          | 12.81            | 16,250                   | 20,000                  |
| 4-5    | 109,000     | 127,570           | 10.12            | 12,600                   | 14,300                  |
| 5-6    | -122,500    | -136,080          | 8.37             | 16,300                   | 20,000                  |
| 6-7    | 14,600      | 24,450            | 5.72             | 4,280                    | 12,500                  |
| 7-8    | -150,000    | -188,700          | 10.61            | 17,750                   | 20,000                  |
| 8-9    | 94,500      | 124,600           | 11.86            | 10,500                   | 15,400                  |
| 9-10   | -171,000    | -206,700          | 11.72            | 17,650                   | 20,000                  |
| 10-10  | -31,000     | -31,000           | 3.46             | 8,950                    | 20,000                  |

— 1 —

the 1990s, the number of people in the world who are under 15 years of age is expected to increase from 1.1 billion to 1.5 billion. The number of people aged 65 and over is expected to increase from 200 million to 400 million. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion.

[illegible]

1. The first step in the process is to identify the problem or issue that needs to be addressed. This involves gathering information and understanding the context of the problem.

2. Once the problem is identified, the next step is to define the objectives and goals of the project. This helps to clarify what needs to be achieved and provides a clear direction for the work.

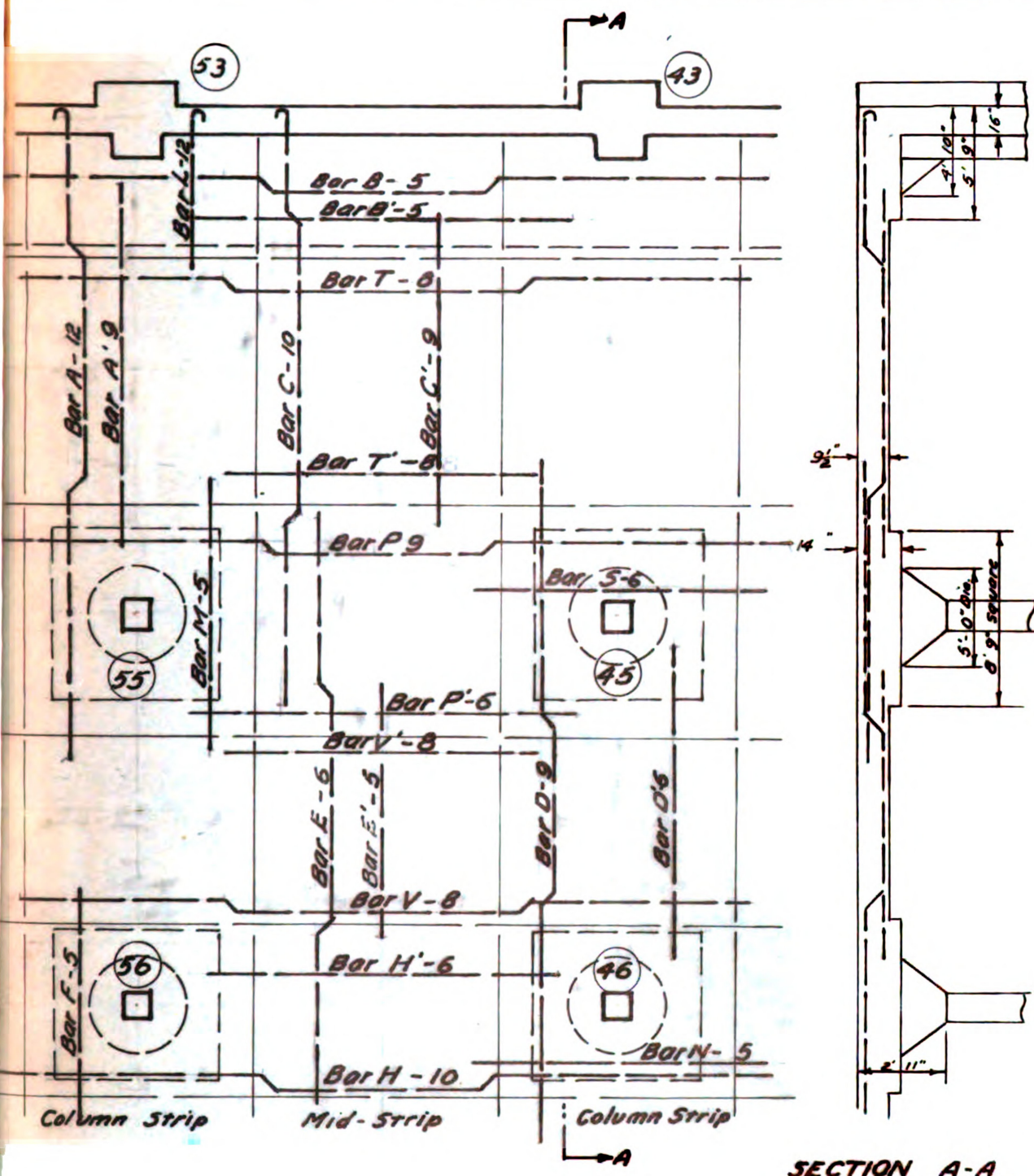
3. The third step is to develop a plan or strategy to address the problem. This involves breaking down the problem into smaller, manageable tasks and determining the resources needed to complete them.

4. The fourth step is to implement the plan. This involves putting the strategy into action and monitoring progress to ensure that the objectives are being met.

5. Finally, the fifth step is to evaluate the results of the project. This involves assessing the outcomes against the objectives and identifying any areas for improvement or further action.

[illegible][illegible][illegible]



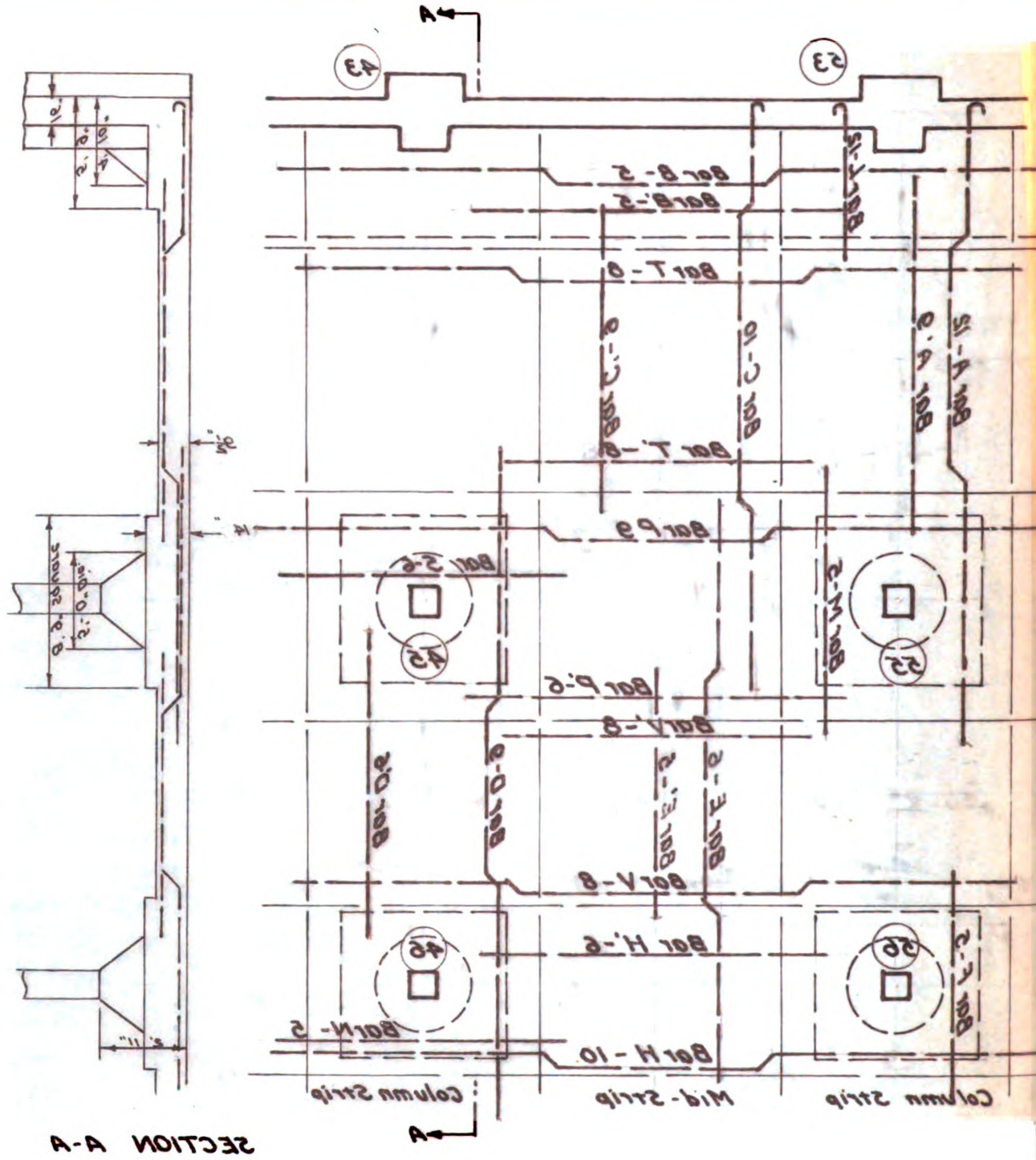


SECTION A-A

SECTION OF FLOOR PLAN MAIN AUDITORIUM  
 SHOWING TYPICAL INTERIOR & EXTERIOR PANELS  
 SCALE ~  $\frac{1}{8}" = 1'-0"$   
 Note - Reinforcing Bars Repeat in Similar Strips



SECTION OF FLOOR PLAN MAIN AUDITORIUM  
 SHOWING TYPICAL INTERIOR & EXTERIOR PANELS  
 SCALE -  $\frac{1}{8}" = 1'-0"$   
 Note - Reinforcing Bars Repeat in Similar Strips



Reinforcing,  $111.1 + 100 + 10 = 221.1$  sq. ft.  
 A slab of this thickness, 221.1 sq. ft. will be OK.

The total thickness  $T_2$  of the slab shall be not less than that given by the following;

$T_2 = .08 \times l \sqrt{W'} + 1$  for 2' sq. ft. 2000 pounds/sq. inch where  $l$  = length along side of panel and  $W'$  = sq. ft. of uniformly distributed loads.

For ultimate strength concrete, multiply the results by the factor  $\sqrt{\frac{2000}{f'_c}}$ .

The concrete is to have an ultimate strength of  $f'_c = 2000$  pounds/sq. inch.

The required thickness:

$$T_2 = (.08 \times 25 \times 12 \sqrt{200} + 1) \sqrt{\frac{2000}{2000}} \\ = 6.4 \text{ in. as compared to 11 in. provided.}$$

For purposes of analysis the slab is considered as consisting of strips, each one half panel in width. The column strip is symmetrical about the column center line and the middle strip about the panel center line. The critical sections to be analyzed for moments are as follows:

Negative moments - Edges of the panel, lines joining column centers and lines following the perimeter of the capital.

Positive moments - Center lines of all panels.

Using the above referencing, the numerical values of the positive and negative bending moments in the direction of either side of the panel is:

$$M_o = 0.09 Wl \left(1 - \frac{2\beta}{\beta_1}\right)^2$$

Where  $l$  is the length in feet in the direction considered and  $W$  the weight of the panel in pounds, the moments for the sections to be analyzed are as a function of  $W$  and the coefficients used are given by the A. C. I.

#### INERTIA MOMENT:

East and west directions

$$W = 25 \times 20 \times 150 = 125,000 \text{ pounds}$$

$$M_o = .09 \times 125,000 \times 25 \left(1 - \frac{2/3 \times 5/15}\right)^2 = 210,000 \text{ ft.-lbs.} \\ = 2,520,000 \text{ in.-lbs.}$$

In the North and South direction:

$$M_o = .09 \times 125,000 \times 20 \left(1 - \frac{2/3 \times 5/20}\right)^2 = 150,000 \text{ ft.-lbs.} \\ = 1,875,000 \text{ in.-lbs.}$$





To simplify checking the design, negative moment in any column strip = 0.50  $M_o$ .

$$M_o = 0.80 \times 2,500,000 = 1,999,999.0 \text{ in.-lbs}$$

This is for a beam of width  $b = 0.75 \times 4.38' = 3.285'$  and height  $d = T - 1\frac{1}{2} = 15\frac{1}{2}$  inches.

$$K = \frac{M_o}{b d^2} = \frac{1,999,999}{0.75 \times 12 \times 15.5 \times 15.5} = 72$$

From Table:  $f_c = 600$  pounds/sq. inches-safe.

#### DIAGONAL TENSION AND SHEAR:

$V = \frac{V}{b d}$  where  $V$  equals the maximum shearing stress.

For an interior column the critical section is a square whose sides are equal to  $12(3\frac{1}{2}) + 2 \times (t-1.5)$  121 inches.

$$V = \frac{(25 \times 80 - \frac{121 \times 121}{144}) 250}{4 \times 121 \times 7/8 \times 8} = \frac{93,500}{454 \times 7} = 29 \text{ lbs./sq. in.}$$

$$\text{Allowable } V = .03 f_c' b d = 75 \text{ lbs./sq. inch}.$$

#### BOND STRENGTH:

The bond requirements are taken care of by detailed specifications for minimum bar lengths, points of inflection and percentage of steel to be extended through as bent up. The actual lengths and points of bending bars are given in the tables and compared with the A. C. I. requirements.

#### INTERIOR PANEL

Steel area requirements are tabulated as for the interior panel. The bond requirements are also included in tables.

#### EXTERIOR PANEL - E & W

|                       | Column<br>Positive | Strip<br>Negative | Mid<br>Positive | Strip<br>Negative |
|-----------------------|--------------------|-------------------|-----------------|-------------------|
| Moment<br>Coefficient | 0.80               | 0.50              | 0.15            | 0.15              |
| Moment<br>In.-lbs.    | 504,000            | 1,999,999         | 331,000         | 331,000           |
| Steel<br>provided     | 15-5/8"            | 21-5/8"           | 16-1" sq.       | 16-1" sq.         |

Depth  $d$ , in.      6"      18.5"      8"      8"

|                                             | Column<br>Positive | Strip<br>Negative | Wall<br>Positive | Strip<br>Negative |
|---------------------------------------------|--------------------|-------------------|------------------|-------------------|
| As = $\frac{M}{20,000 \times 7/8 \times 1}$ | 3.88 sq."          | 5.70 sq."         | 3.72 sq."        | 3.72 sq."         |
| As provided                                 | 4.52 sq."          | 7.35 sq."         | 4.00 sq."        | 4.00 sq."         |
| K                                           | 58.5               | 81                | 33.7             | 33.7              |
| Tc                                          | 500                | 350               | 480              | 480               |

# INTERIOR PANEL - E & W (Typical)

|                       | Column<br>Positive         | Strip<br>Negative                                   | Wall<br>Positive      | Strip<br>Negative      |
|-----------------------|----------------------------|-----------------------------------------------------|-----------------------|------------------------|
| Moment<br>Coefficient | 0.80                       | 0.50                                                | 0.15                  | 0.15                   |
| Moment<br>in.-lbs.    | 501,000                    | -1,330,000                                          | 301,000               | -381,000               |
| Steel<br>Provided     | 10- $\frac{1}{4}$ " sq. BT | 5- $\frac{1}{4}$ " sq. in top 3- $\frac{1}{4}$ " BT |                       |                        |
|                       | 3- $\frac{1}{4}$ " sq. ST  | 20- $\frac{1}{4}$ " sq. ST                          | 3- $\frac{1}{4}$ " ST | 10- $\frac{1}{4}$ " ST |

|                                             |             |              |              |              |
|---------------------------------------------|-------------|--------------|--------------|--------------|
| Depth d, in.                                | 6 in.       | 13.5 in.     | 6 in.        | 6 in.        |
| As = $\frac{M}{20,000 \times 7/8 \times 1}$ | 3.8 sq. in. | 5.75 sq. in. | 3.72 sq. in. | 3.72 sq. in. |
| As Provided                                 | 4.0 sq. in. | 6.25 sq. in. | 3.14 sq. in. | 3.14 sq. in. |
| $K = \frac{M}{b d^2}$                       | 64.5        | 72           | 49.3         | 49.3         |
| Tc                                          | 400         | 300 13/8 in. | 480          | 400          |

# EXTERIOR PANEL - E & W

|                       | Column<br>Positive | Strip<br>At Wall<br>Negative | Wall<br>Positive        | Strip<br>At Wall<br>Negative |
|-----------------------|--------------------|------------------------------|-------------------------|------------------------------|
| Moment<br>Coefficient | 0.80x1.25          | 0.50x300                     | .15x1.25                | 0.15x0.125                   |
| Moment<br>in.-lbs.    | 652,000            | -1,155,000                   | 452,000                 | -212,000                     |
| Steel provided        | 22-5/8" R1         | 21-5/8" R1                   | 12- $\frac{1}{4}$ " sq. | 10- $\frac{1}{4}$ " sq.      |

|                                             |              |              |              |              |
|---------------------------------------------|--------------|--------------|--------------|--------------|
| Depth d, in.                                | 7.5 in.      | 12 in.       | 7.5 in.      | 6 in.        |
| As = $\frac{M}{20,000 \times 7/8 \times 1}$ | 4.25 sq. in. | 5.35 sq. in. | 3.80 sq. in. | 1.73 sq. in. |

## TABLE 1.1.1.1

|            | 1970       | 1971       | 1972       | 1973       |
|------------|------------|------------|------------|------------|
| 1.1.1.1.1  | 1.1.1.1.1  | 1.1.1.1.1  | 1.1.1.1.1  | 1.1.1.1.1  |
| 1.1.1.1.2  | 1.1.1.1.2  | 1.1.1.1.2  | 1.1.1.1.2  | 1.1.1.1.2  |
| 1.1.1.1.3  | 1.1.1.1.3  | 1.1.1.1.3  | 1.1.1.1.3  | 1.1.1.1.3  |
| 1.1.1.1.4  | 1.1.1.1.4  | 1.1.1.1.4  | 1.1.1.1.4  | 1.1.1.1.4  |
| 1.1.1.1.5  | 1.1.1.1.5  | 1.1.1.1.5  | 1.1.1.1.5  | 1.1.1.1.5  |
| 1.1.1.1.6  | 1.1.1.1.6  | 1.1.1.1.6  | 1.1.1.1.6  | 1.1.1.1.6  |
| 1.1.1.1.7  | 1.1.1.1.7  | 1.1.1.1.7  | 1.1.1.1.7  | 1.1.1.1.7  |
| 1.1.1.1.8  | 1.1.1.1.8  | 1.1.1.1.8  | 1.1.1.1.8  | 1.1.1.1.8  |
| 1.1.1.1.9  | 1.1.1.1.9  | 1.1.1.1.9  | 1.1.1.1.9  | 1.1.1.1.9  |
| 1.1.1.1.10 | 1.1.1.1.10 | 1.1.1.1.10 | 1.1.1.1.10 | 1.1.1.1.10 |

## TABLE 1.1.1.2

|            | 1970       | 1971       | 1972       | 1973       |
|------------|------------|------------|------------|------------|
| 1.1.1.2.1  | 1.1.1.2.1  | 1.1.1.2.1  | 1.1.1.2.1  | 1.1.1.2.1  |
| 1.1.1.2.2  | 1.1.1.2.2  | 1.1.1.2.2  | 1.1.1.2.2  | 1.1.1.2.2  |
| 1.1.1.2.3  | 1.1.1.2.3  | 1.1.1.2.3  | 1.1.1.2.3  | 1.1.1.2.3  |
| 1.1.1.2.4  | 1.1.1.2.4  | 1.1.1.2.4  | 1.1.1.2.4  | 1.1.1.2.4  |
| 1.1.1.2.5  | 1.1.1.2.5  | 1.1.1.2.5  | 1.1.1.2.5  | 1.1.1.2.5  |
| 1.1.1.2.6  | 1.1.1.2.6  | 1.1.1.2.6  | 1.1.1.2.6  | 1.1.1.2.6  |
| 1.1.1.2.7  | 1.1.1.2.7  | 1.1.1.2.7  | 1.1.1.2.7  | 1.1.1.2.7  |
| 1.1.1.2.8  | 1.1.1.2.8  | 1.1.1.2.8  | 1.1.1.2.8  | 1.1.1.2.8  |
| 1.1.1.2.9  | 1.1.1.2.9  | 1.1.1.2.9  | 1.1.1.2.9  | 1.1.1.2.9  |
| 1.1.1.2.10 | 1.1.1.2.10 | 1.1.1.2.10 | 1.1.1.2.10 | 1.1.1.2.10 |

|            |            |            |            |            |
|------------|------------|------------|------------|------------|
| 1.1.1.3.1  | 1.1.1.3.1  | 1.1.1.3.1  | 1.1.1.3.1  | 1.1.1.3.1  |
| 1.1.1.3.2  | 1.1.1.3.2  | 1.1.1.3.2  | 1.1.1.3.2  | 1.1.1.3.2  |
| 1.1.1.3.3  | 1.1.1.3.3  | 1.1.1.3.3  | 1.1.1.3.3  | 1.1.1.3.3  |
| 1.1.1.3.4  | 1.1.1.3.4  | 1.1.1.3.4  | 1.1.1.3.4  | 1.1.1.3.4  |
| 1.1.1.3.5  | 1.1.1.3.5  | 1.1.1.3.5  | 1.1.1.3.5  | 1.1.1.3.5  |
| 1.1.1.3.6  | 1.1.1.3.6  | 1.1.1.3.6  | 1.1.1.3.6  | 1.1.1.3.6  |
| 1.1.1.3.7  | 1.1.1.3.7  | 1.1.1.3.7  | 1.1.1.3.7  | 1.1.1.3.7  |
| 1.1.1.3.8  | 1.1.1.3.8  | 1.1.1.3.8  | 1.1.1.3.8  | 1.1.1.3.8  |
| 1.1.1.3.9  | 1.1.1.3.9  | 1.1.1.3.9  | 1.1.1.3.9  | 1.1.1.3.9  |
| 1.1.1.3.10 | 1.1.1.3.10 | 1.1.1.3.10 | 1.1.1.3.10 | 1.1.1.3.10 |

# TABLE 1. SUMMARY OF DATA

| Run | Time  | Location | Depth | Speed | Direction |
|-----|-------|----------|-------|-------|-----------|
| 1   | 10:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 2   | 10:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 3   | 10:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 4   | 10:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 5   | 10:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 6   | 10:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 7   | 10:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 8   | 10:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 9   | 10:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 10  | 10:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 11  | 10:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 12  | 10:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 13  | 11:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 14  | 11:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 15  | 11:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 16  | 11:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 17  | 11:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 18  | 11:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 19  | 11:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 20  | 11:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 21  | 11:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 22  | 11:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 23  | 11:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 24  | 11:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 25  | 12:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 26  | 12:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 27  | 12:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 28  | 12:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 29  | 12:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 30  | 12:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 31  | 12:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 32  | 12:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 33  | 12:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 34  | 12:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 35  | 12:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 36  | 12:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 37  | 13:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 38  | 13:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 39  | 13:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 40  | 13:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 41  | 13:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 42  | 13:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 43  | 13:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 44  | 13:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 45  | 13:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 46  | 13:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 47  | 13:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 48  | 13:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 49  | 14:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 50  | 14:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 51  | 14:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 52  | 14:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 53  | 14:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 54  | 14:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 55  | 14:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 56  | 14:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 57  | 14:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 58  | 14:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 59  | 14:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 60  | 14:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 61  | 15:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 62  | 15:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 63  | 15:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 64  | 15:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 65  | 15:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 66  | 15:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 67  | 15:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 68  | 15:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 69  | 15:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 70  | 15:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 71  | 15:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 72  | 15:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 73  | 16:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 74  | 16:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 75  | 16:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 76  | 16:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 77  | 16:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 78  | 16:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 79  | 16:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 80  | 16:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 81  | 16:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 82  | 16:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 83  | 16:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 84  | 16:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 85  | 17:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 86  | 17:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 87  | 17:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 88  | 17:15 | 100.0    | 10.0  | 10.0  | 10.0      |
| 89  | 17:20 | 100.0    | 10.0  | 10.0  | 10.0      |
| 90  | 17:25 | 100.0    | 10.0  | 10.0  | 10.0      |
| 91  | 17:30 | 100.0    | 10.0  | 10.0  | 10.0      |
| 92  | 17:35 | 100.0    | 10.0  | 10.0  | 10.0      |
| 93  | 17:40 | 100.0    | 10.0  | 10.0  | 10.0      |
| 94  | 17:45 | 100.0    | 10.0  | 10.0  | 10.0      |
| 95  | 17:50 | 100.0    | 10.0  | 10.0  | 10.0      |
| 96  | 17:55 | 100.0    | 10.0  | 10.0  | 10.0      |
| 97  | 18:00 | 100.0    | 10.0  | 10.0  | 10.0      |
| 98  | 18:05 | 100.0    | 10.0  | 10.0  | 10.0      |
| 99  | 18:10 | 100.0    | 10.0  | 10.0  | 10.0      |
| 100 | 18:15 | 100.0    | 10.0  | 10.0  | 10.0      |



## B.M. AND SLAB SYSTEM- WEST END

The slabs resting on beams 101 and 103 is 4' thick and has 3/8" Rd. bars 8" on centers for maximum bending.

$$\text{Wt. of slab} = \frac{4 \times 150}{12} = 50 \text{ lbs/ sq. ft.}$$

$$\text{Live load} \quad \frac{70}{120} \text{ lbs/ sq. ft.}$$

The slab will be considered partially fixed.  
Consider one foot width.

$$\text{Max. B.M.} = \frac{w l^2}{8} = \frac{120 \times (4.75)^2}{8} = 338 \text{ ft. lbs.}$$

$$K = \frac{338 \times 12}{12 \times (2.5)^2} = 54 \quad p = \frac{.16}{2.5 \times 12} = .0053$$

$$f_c = 400 \text{ lb./sq. in.} \quad f_s = 11,400 \text{ lb./sq. in.}$$

$$V = 120 \times 4.75 = 285 \text{ lbs.}$$

$$v = \frac{V}{b j d} = \frac{285}{12 \times 7/8 \times 2.5} = 10.9 \text{ lb./sq. in.}$$

$$U = \frac{V}{2 b j d} = \frac{285}{2 \times 12 \times 7/8 \times 2.5} = 73.6 \text{ lb./sq. in.}$$

The slab supported by beams 101, 102, and the N.W. corner is 5" thick and has 3/8" rd bars 8" o.c. for positive bending.

$$\text{Wt. of slab} = \frac{5 \times 150}{12} = 62.5$$

$$\text{Live load} \quad \frac{70}{132.5} \text{ lbs/ sq.ft.}$$

This slab is partially fixed on either end.

$$\text{Max. B.M.} = \frac{w l^2}{8} = \frac{132.5 \times 64^2}{8} = 1060 \text{ ft.lbs.}$$

$$K = \frac{1060 \times 12}{12 \times 3.5 \times 3.5} = 86.5 \quad p = \frac{.3}{12 \times 3.5} = .0071$$

$$f_c = 530 \text{ lbs./sq. in.} \quad f_s = 13,900 \text{ lbs./sq. in.}$$

$$V = \frac{132.5 \times 8}{2} = 530.0 \text{ lbs.}$$

$$v = \frac{530}{12 \times 7/8 \times 3.5} = 14.4 \text{ lbs./sq. in.}$$

$$U = \frac{530}{2 \times 12 \times 7/8 \times 3.5} = 73.8 \text{ lbs./sq.in.}$$

Of the two beams 101, the one carrying the heaviest load will be investigated. The beam will be considered rectangular in shape, 10" x 14". The steel provided is 2-3/4" dia. straight bars and 1-5/8" rd. bend bar. Clear upon is 12 ft.

$$\text{Slab load} = 4 \times 132.5 = 530 \text{ lbs./ft.}$$

$$\text{Wt. of stem} = \frac{10 \times 130}{14} = 146 \text{ lbs./ft.}$$

$$\text{Total load} \quad 676 \text{ lbs./ft.}$$

28

104  
13

29

14

30

102  
103  
101  
4" Slab  
3/8"  $\phi$  8" o.c.  
3/8"  $\phi$  12" o.c.  
5" Slab  
1/2"  $\phi$  8" o.c.  
3/8"  $\phi$  12" o.c.

108  
107  
106  
109  
32A  
8" Slab  
5/8"  $\phi$  6" o.c.  
Both Ways  
105  
4" Slab  
3/8"  $\phi$  6" o.c.  
3/8"  $\phi$  12" o.c.

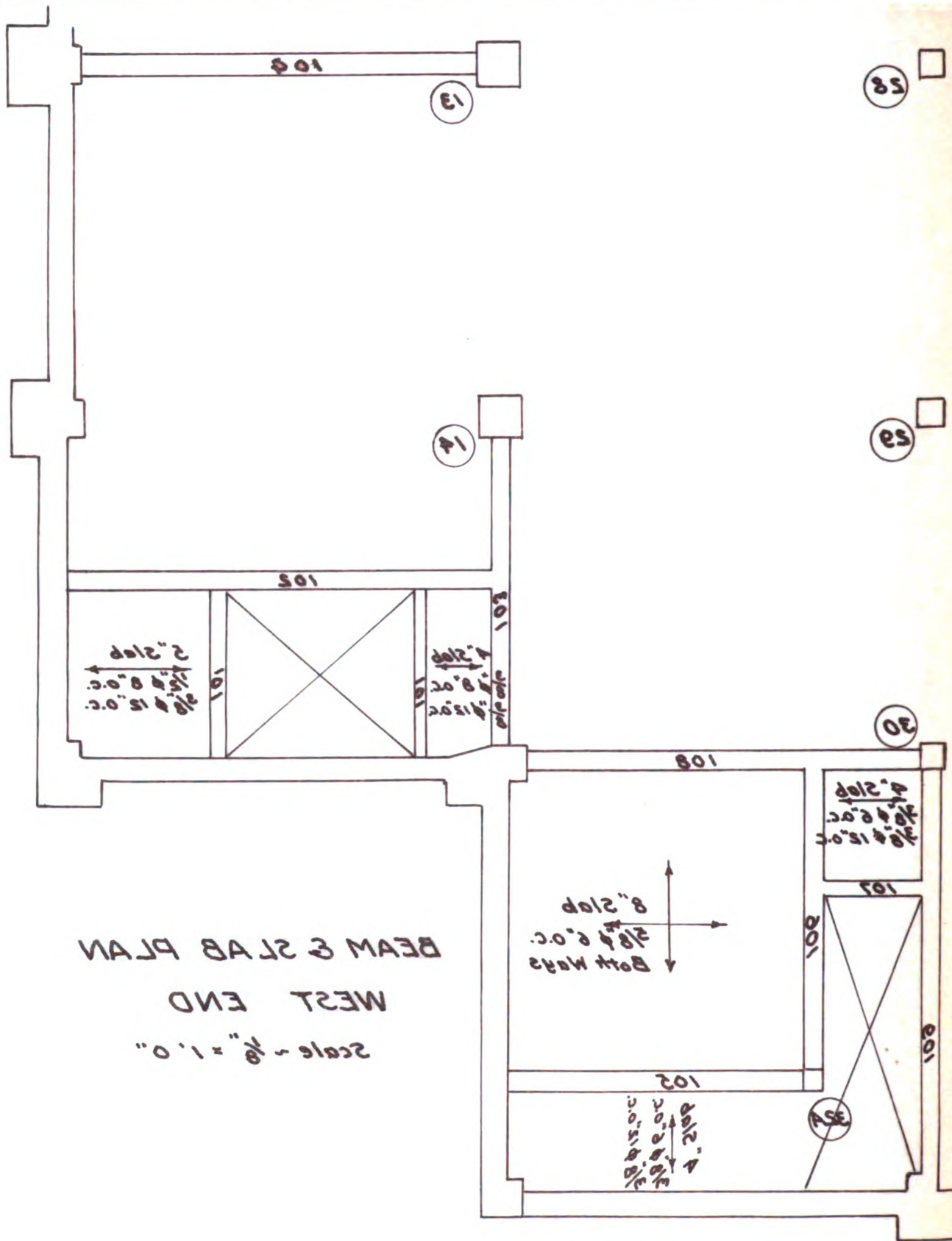
BEAM & SLAB PLAN

WEST END

Scale ~ 1/8" = 1' 0"

# BEAM & SLAB PLAN WEST END

Scale -  $\frac{1}{8}" = 1' 0"$





$$\text{Max. B.M.} = \frac{676 \times 9.5 \times 9.5}{6} = 7,630 \text{ ft. lbs.}$$

$$K = \frac{7630 \times 12}{10 \times 12 \times 12} = 63.5 \quad p = \frac{1.19}{12.10} = .0099$$

$$K = \frac{2\sqrt{2np + (np)^2} - np}{12.10} = \frac{\sqrt{2 \times 1.19 \times .0099 + (.0099)^2} - .0099}{12.10} = .39$$

$$j = 1 - \frac{K}{3} = .87 \quad jd = 10.4"$$

$$fs = \frac{M}{Asjd} = \frac{7630 \times 12}{1.19 \times 10.4} = 7,400 \text{ lbs./sq. in.}$$

$$fc = \frac{2M}{Kjbd^2} = \frac{7630 \times 12 \times 2}{.39 \times .87 \times 10 \times 12 \times 12} = 365 \text{ lbs./sq. in.}$$

$$V = \frac{676 \times 9.75}{2} = 3780 \text{ lbs.}$$

$$v = \frac{3780}{10 \times 10.4} = 36.3 \text{ lbs/sq. in.}$$

$$u = \frac{3780}{4.71 \times 10.4} = 77 \text{ lbs./sq. in.}$$

### Beam No. 102

This beam carries the two concentrated reactions from beams 101 plus part of the load transmitted from the  $9\frac{1}{2}"$  flat slab. The beam is 12" x 24" and rectangular.

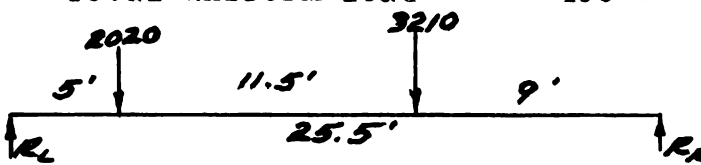
The concentrated loads are

$$676 \times 4.75 = 3210 \text{ lbs. and } 450 \times 4.75 = 2040 \text{ lbs.}$$

$$\text{Live load} \quad 100$$

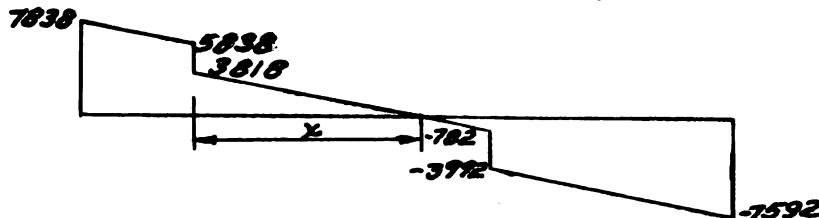
$$\text{Slab} = \frac{12 \times 24 \times 150}{144} = 300$$

$$\text{Total uniform load} \quad 400 \text{ #/'}$$



$$R_1 = \frac{400 \times 25.5}{2} + \frac{9}{25.5} \times 3210 + \frac{20.5}{25.5} \times 2020 = 7837 \text{ lbs.}$$

$$R_2 = \frac{400 \times 25.5}{2} + \frac{5}{25.5} \times 2020 + \frac{16.5}{25.5} \times 3210 = 7592 \text{ lbs.}$$



$$x = \frac{3658 \times 11.5}{4600} = 9.65'$$

$$\text{Max. B.M.} = \frac{7838 \times 5}{2} + \frac{9.65 \times 3658}{2} = 52,850 \text{ ft. lbs.}$$

$$K = \frac{52,850 \times 12}{12 \times 22 \times 22} = 110 \quad p = \frac{5.34}{22 \times 12} = .021$$

$$\text{From plots: } j = .84 \quad K = .46$$

$$fs = \frac{52,850 \times 12}{12 \times .84 \times 5.34} = 11,800 \text{ #/sq. in.}$$

$$V = \frac{113 + 113}{2} \times 17.5 = 1975 \text{ lbs.}$$

$$u = \frac{1975}{18 \times 11.5} = 94 \text{ lbs./sq. in.}$$

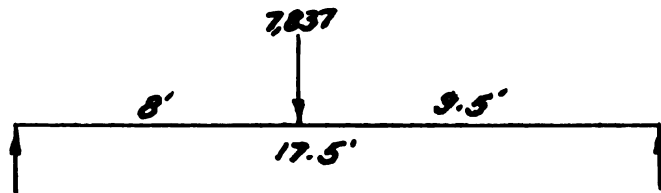
113

113 lbs. 57 lbs. 57 lbs. 3-1 1/8" sq. 3-1 1/8" sq. 3-1 1/8" sq.

113 lbs. 57 lbs. 57 lbs. 3-1 1/8" sq. 3-1 1/8" sq. 3-1 1/8" sq.

$$\text{113 load} = (113 + 113) \times 11.5 = 2590$$

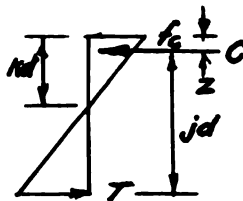
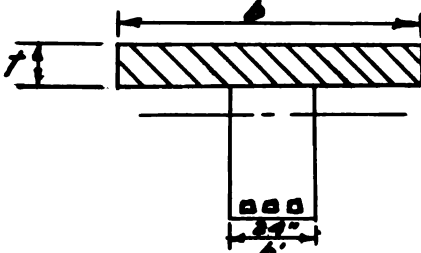
$$\text{Total load} = \frac{2590}{3.15} = 820 \text{ lbs./sq.}$$



$$1. \text{ M. due to uniform load} = \frac{wL^2}{8} = \frac{820 \times (17.5)^2}{8} = 250,000$$

$$2. \text{ M. due to c.c. load} = \left( \frac{2837}{17.5} \times 7.5 \right) \times 9.5 = 113,000 \text{ lbs.}$$

$$\text{Total M. M.} = 363,000 \text{ ft. lbs.}$$



$$Kc = \frac{250,000 + 113,000}{27 \times 18 \times 11.5} = \frac{363,000}{558} = 6.5$$

$$u = \frac{7(18 - 0.7)}{8(18 - 1)} = \frac{1(17.3)}{14} = 1.24$$

$$jd = 11.5 - 1.24 = 10.26 \quad j = 0.89$$

$$fc = \frac{363,000}{18 \times 11.5 \times 10.26} = 18,000 \text{ lbs./sq. in.}$$

$$T = \frac{363,000}{18 \times 11.5 \times 10.26} \times \left( \frac{18}{18 - 0.7} \right) = 300 \text{ lbs./sq. in.}$$

$$V = 363,000 \times 0.75 + 113,000 \times 0.5 = 31,000$$

$$v = \frac{31,000}{18 \times 11.5} = 148.5 \text{ lbs./sq. in. O.K.}$$

$$u = \frac{31,000}{18 \times 11.5} = 148.5 \text{ lbs./sq. in. O.K.}$$

This value for load is 148.5 lbs./sq. in. 148.5 lbs./sq. in. 148.5 lbs./sq. in.

148.5 lbs./sq. in. 148.5 lbs./sq. in. 148.5 lbs./sq. in. 148.5 lbs./sq. in. 148.5 lbs./sq. in.

Bars 6 $\frac{1}{2}$  o.c. Clear dimensions are 17' x 17'

For a one foot strip in either direction the uniform load is  $w$ .  $d$  is taken =  $8 - 1.5 = 6.5"$

Live load  $\frac{80}{3}$  lbs./sq. ft.

Slab  $\frac{8}{12} \times 150 = 100$  lbs./sq. ft.

Total load = 180 lbs./sq. ft.

Max B. M.  $\frac{180 \times 17^2}{8} = 2170$  ft. lbs.

$K = \frac{2170 \times 12}{12 \times 6.5} = 51.3$   $p = \frac{.61}{6.5 \times 12} = .0078$

$j = 0.88$   $k = .35$

$f_s = \frac{2170 \times 12}{.61 \times .88 \times 6.5} = 7,450$

$f_c = \frac{f_s}{n} \left( \frac{K}{1-k} \right) = \frac{7,450 \times .35}{12 \times .65} = 336$  lbs./sq. in.

$v = \frac{60 \times 17}{12 \times .88 \times 6.5} = 14.9$  lbs./sq. in.

$u = \frac{60 \times 17}{5.93 \times .88 \times 6.5} = 45.4$  lbs./sq. in.

Beam No. 105 is an irregular tee section. The maximum effective flange width is  $12 + \frac{17 \times 12}{12} = 29"$ . The

stem is 12" wide and the total  $h = 18"$   $\uparrow = 8"$

Load from 4" slab =  $2.87 (50 + 80) = 374$

Load from 8" slab =  $8.5 (100 + 80) = 1530$

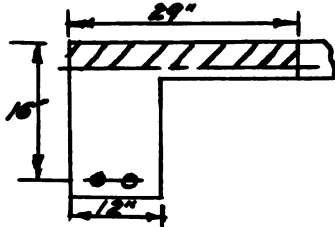
Stem =  $\frac{12 \times 18 \times 150}{144} = 225$

$2,129$  lbs./ft.

Max. B. M. =  $\frac{2129 \times 17^2}{8} = 76,800$  ft. lbs.

$K_d = \frac{2 \times 12 \times 16 \times 3.2 \times 29 \times 64}{2 \times 12 \times 3.2 \times 29 \times 8} = 5.7"$

Since the neutral axis lies along the stem, the beam must therefore be considered rectangular with  $b = 29"$ .



$p = \frac{3.2}{16 \times 29} = .0092$   $k = 0.37$   $j = 0.875$

$f_s = \frac{76,800 \times 12}{.875 \times 16 \times 3.2} = 19,250$  lbs./sq. in.

$f_c = \frac{19,250}{12} \left( \frac{.37}{.63} \right) = 945$  lbs./sq. in.

These fiber stresses are close to the allowable. However, since the 4" slab was not considered as a part of the beam, they are probably not excessive.

$v = \frac{V}{b \cdot d} = \frac{2,129 \times 8.5}{12 \times .875 \times 16} = 107.5$  lbs./sq. in.

$$= \frac{1.5 \times 10^6 \times 1.5}{1.5 \times 10^6 \times 1.5} = 1.5 \times 10^6 / 1.5 \times 10^6$$

The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi.

The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi.

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The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi.

$$d = \frac{1.5 \times 10^6 \times 1.5}{1.5 \times 10^6 \times 1.5} = 1.5 \times 10^6 / 1.5 \times 10^6$$

The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi. The steel stress is 15,000 psi.

$$\begin{aligned} \text{Load } P &= 1.5 \times 10^6 \text{ lbs./ft.} \\ \text{Load } P &= 1.5 \times 10^6 \text{ lbs./ft.} \\ \text{Load } P &= 1.5 \times 10^6 \text{ lbs./ft.} \\ \text{Load } P &= 1.5 \times 10^6 \text{ lbs./ft.} \end{aligned}$$

$$\text{Load } P = \frac{1.5 \times 10^6 \times (1.5)^2}{1.5 \times 10^6 \times 1.5} = 1.5 \times 10^6 / 1.5 \times 10^6$$

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$$V = \frac{1.5 \times 10^6 \times 1.5}{1.5 \times 10^6 \times 1.5} = 1.5 \times 10^6 / 1.5 \times 10^6$$

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4. The fourth part of the document provides a detailed overview of the different types of data that can be collected. It includes information on quantitative data, qualitative data, and mixed-method data. It also discusses the advantages and disadvantages of each type of data and how to choose the most appropriate method for a given study.

5. The fifth part of the document discusses the ethical considerations that must be taken into account when conducting research. It includes information on how to obtain informed consent from participants, how to protect the confidentiality of the data, and how to ensure that the research is conducted in a fair and unbiased manner.

6. The sixth part of the document provides a summary of the key findings of the study. It includes information on the overall results of the research, the limitations of the study, and the implications of the findings for future research. It also discusses the potential for the research to be applied in practice.

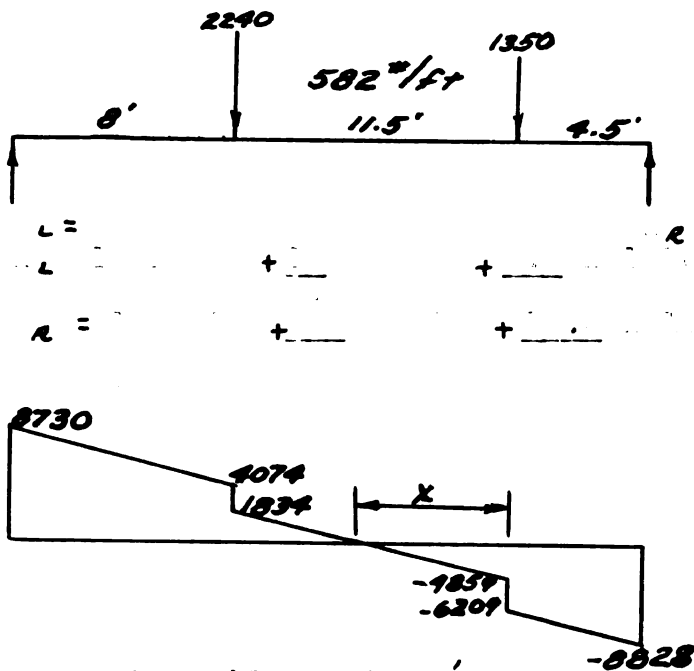
7. The seventh part of the document provides a list of references for the sources used in the study. It includes information on books, articles, and other sources that have been consulted during the research process.

8. The eighth part of the document provides a list of appendices that contain additional information related to the study. This includes information on the survey instrument, the interview schedule, and the data analysis software used.

9. The ninth part of the document provides a list of glossary terms that are used throughout the document. This includes definitions for key concepts and terms that may be unfamiliar to some readers.

10. The tenth part of the document provides a list of acknowledgments that recognize the contributions of the individuals and organizations that have supported the research. This includes information on the funding sources, the research assistants, and the advisors who have provided guidance and support throughout the project.

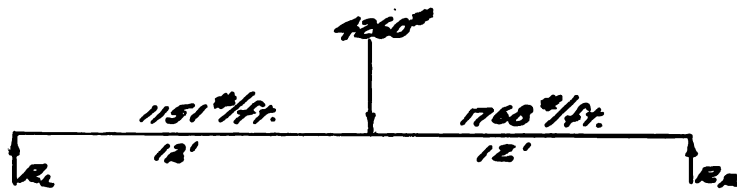




B.M. =

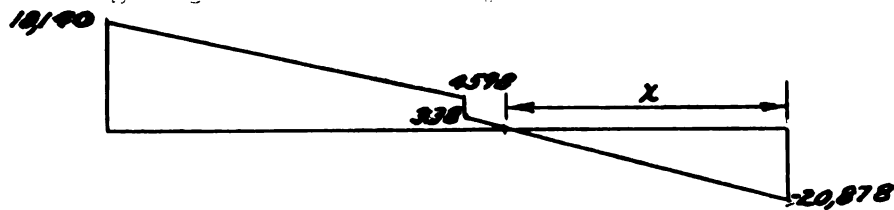
$$+ \frac{1}{2} \times 1151 \times 12 = 6906 \text{ ft.-lb.}$$

$$+ \frac{1}{2} \times 1768 \times 12 = 10608 \text{ ft.-lb.}$$



$$L = \frac{1}{2} \times 1151 \times 12 + \frac{1}{2} \times 1768 \times 12 = 17514 \text{ ft.-lb.}$$

$$R = \frac{1}{2} \times 1151 \times 12 + \frac{1}{2} \times 1768 \times 12 = 17514 \text{ ft.-lb.}$$



$$= \frac{1}{2} \times 1151 \times 12 = 6906 \text{ ft.-lb.}$$

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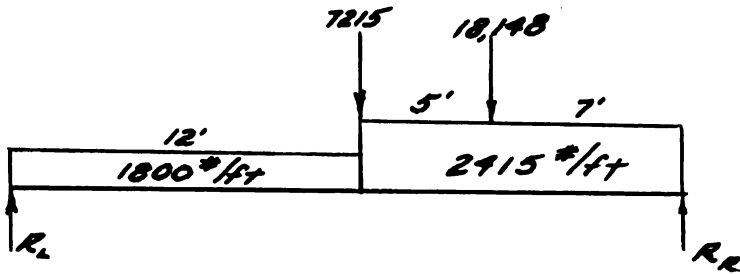
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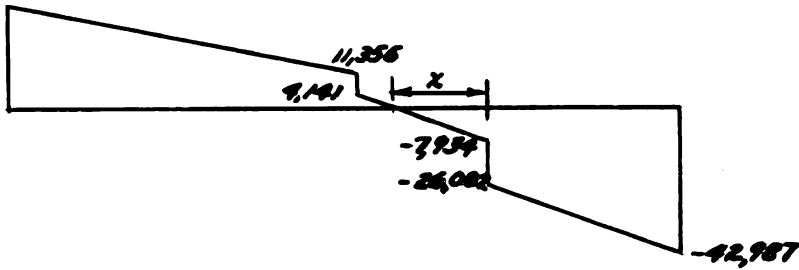
$$= \frac{1}{2} \times 1768 \times 12 = 10608 \text{ ft.-lb.}$$

$$= \frac{1}{2} \times 1151 \times 12 = 6906 \text{ ft.-lb.}$$



$$L = \frac{1}{2} \times 12' \times 1800 \text{ #/ft} + \frac{1}{2} \times 12' \times 2415 \text{ #/ft} = 18,148$$

$$R = \frac{1}{2} \times 12' \times 1800 \text{ #/ft} + \frac{1}{2} \times 12' \times 2415 \text{ #/ft} = 18,148$$



$$= \frac{1}{2} \times 12' \times 1800 \text{ #/ft} = 10,800$$

$$= \frac{1}{2} \times 12' \times 2415 \text{ #/ft} = 14,490$$

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Shear Force Diagram

The shear force diagram is a trapezoid with a top value of 11,356 and a bottom value of -42,987. The width is 12'. The area is 9,141. The diagram is divided into two sections by a vertical line at distance  $x$  from the left end.

$$= \frac{1}{2} \times 12' \times 1800 \text{ #/ft} = 10,800$$

$$= \frac{1}{2} \times 12' \times 2415 \text{ #/ft} = 14,490$$

$$= \frac{1}{2} \times 12' \times 1800 \text{ #/ft} = 10,800$$

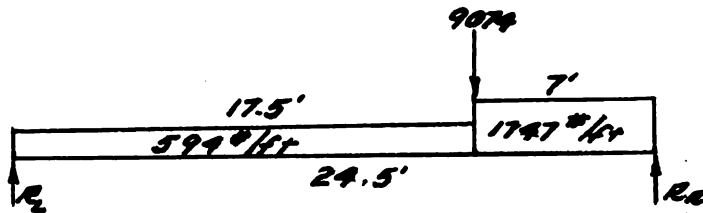
$$= \frac{1}{2} \times 12' \times 2415 \text{ #/ft} = 14,490$$

$$= \frac{1}{2} \times 12' \times 1800 \text{ #/ft} = 10,800$$

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$$= \frac{1}{2} \times 12' \times 1800 \text{ #/ft} = 10,800$$

$$S = \frac{1000 \times 1000}{1000} = 1000 \text{ ft.}^2$$

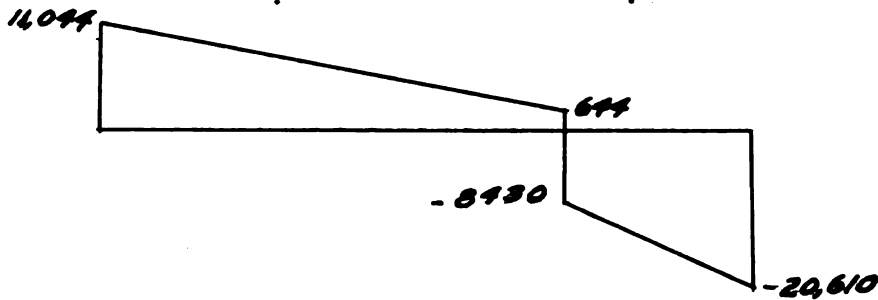


$$50 + \frac{1000 \times 1000}{1000} + \frac{1000 \times 1000}{1000} = 1000 \text{ ft.}^2$$

$$50 + \frac{1000 \times 1000}{1000} = 1000 \text{ ft.}^2$$

$$A_L = (1000 \times 1000) \frac{1}{1000} + (1000 \times 1000) \frac{1}{1000} + \frac{1000 \times 1000}{1000} = 1000 \text{ ft.}^2$$

$$R = (1000 \times 1000) \frac{1}{1000} + (1000 \times 1000) \frac{1}{1000} + \frac{1000 \times 1000}{1000} = 1000 \text{ ft.}^2$$



$$1000 \times 1000 = 1000 \text{ ft.}^2$$

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1. The first step is to identify the variables involved in the problem.

2. Next, we need to determine the relationship between these variables.

3. Once the relationship is established, we can use it to solve for the unknowns.

4. Finally, we should check our solution to ensure it makes sense in the context of the problem.

$$\begin{array}{r} 100 \\ 100 \\ \hline \text{Total} = 200 \end{array}$$

$$100 \times 2 = 200$$

5. The next step is to calculate the total value of the items.

$$100 = 100$$

$$100 = 100$$

$$100 = 100$$

$$100 = 100$$

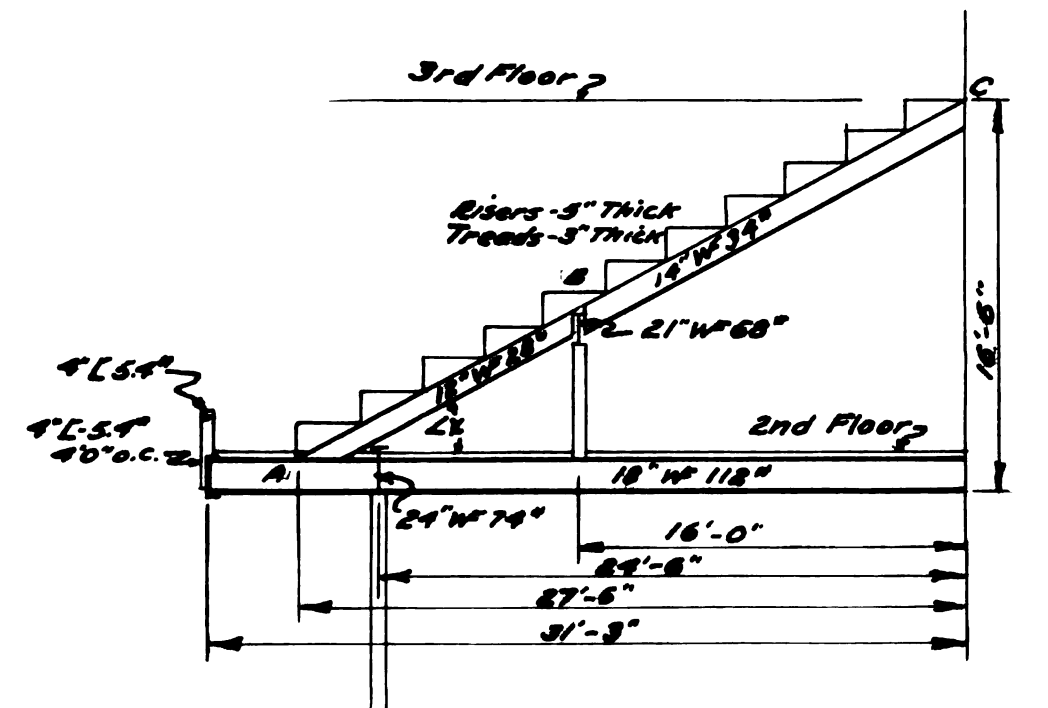
$$100 = 100$$

6. The final step is to verify the solution.

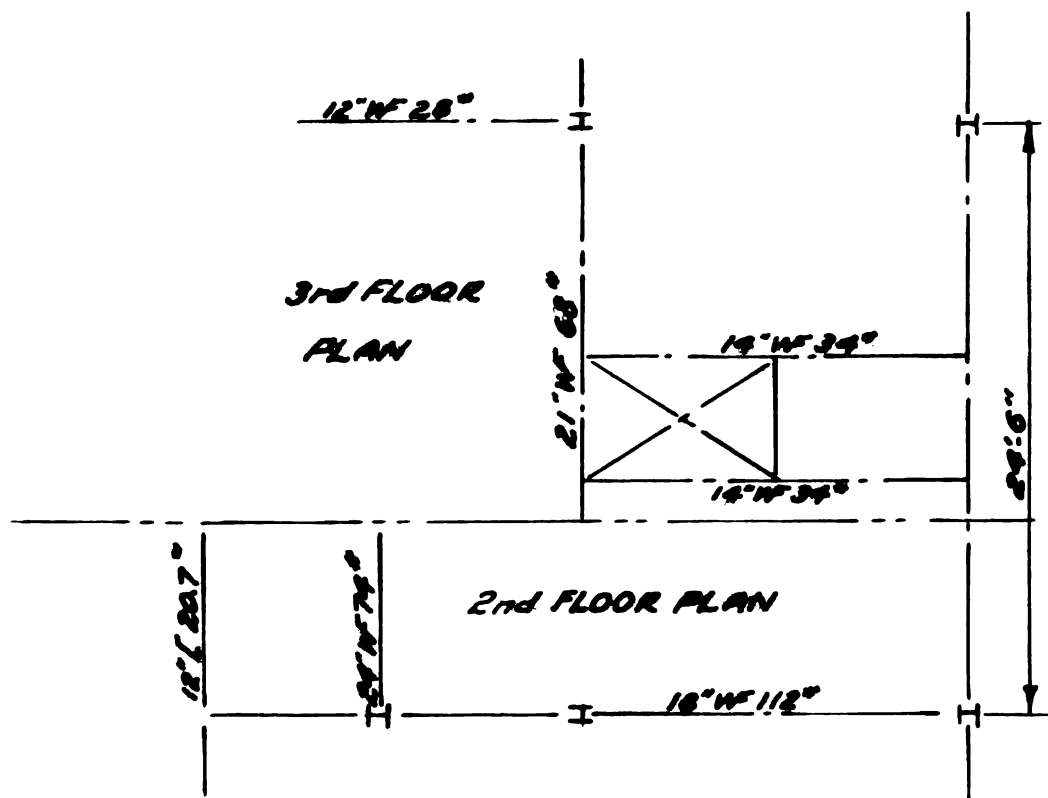
7. The total value of the items is 200.

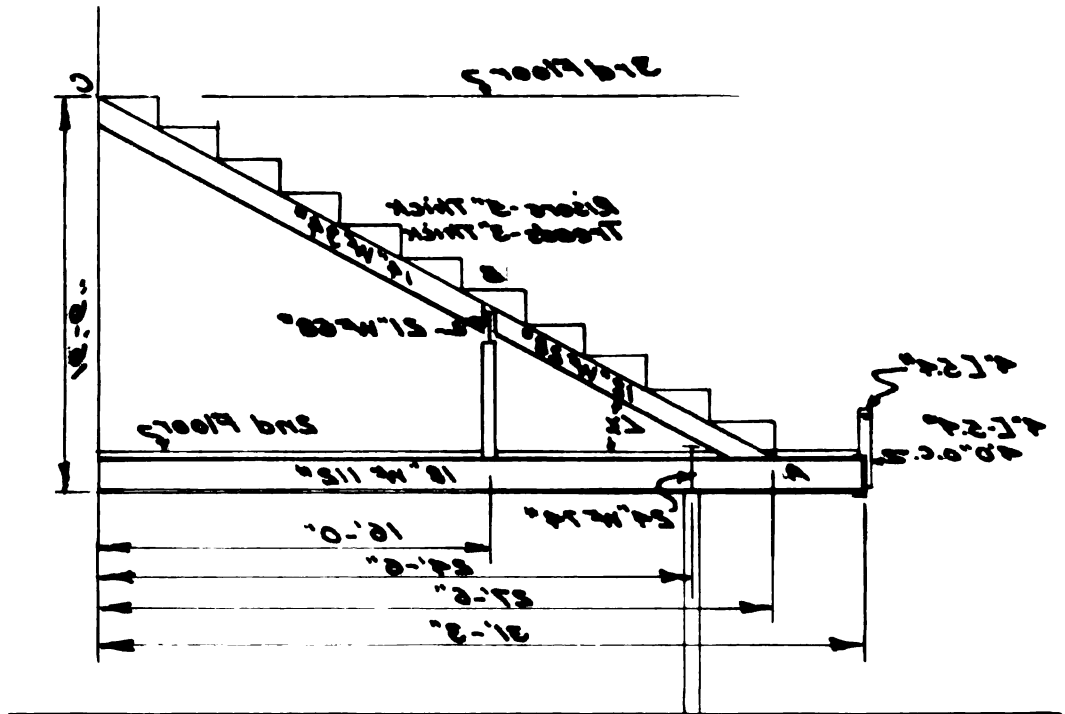
8. The solution is verified.



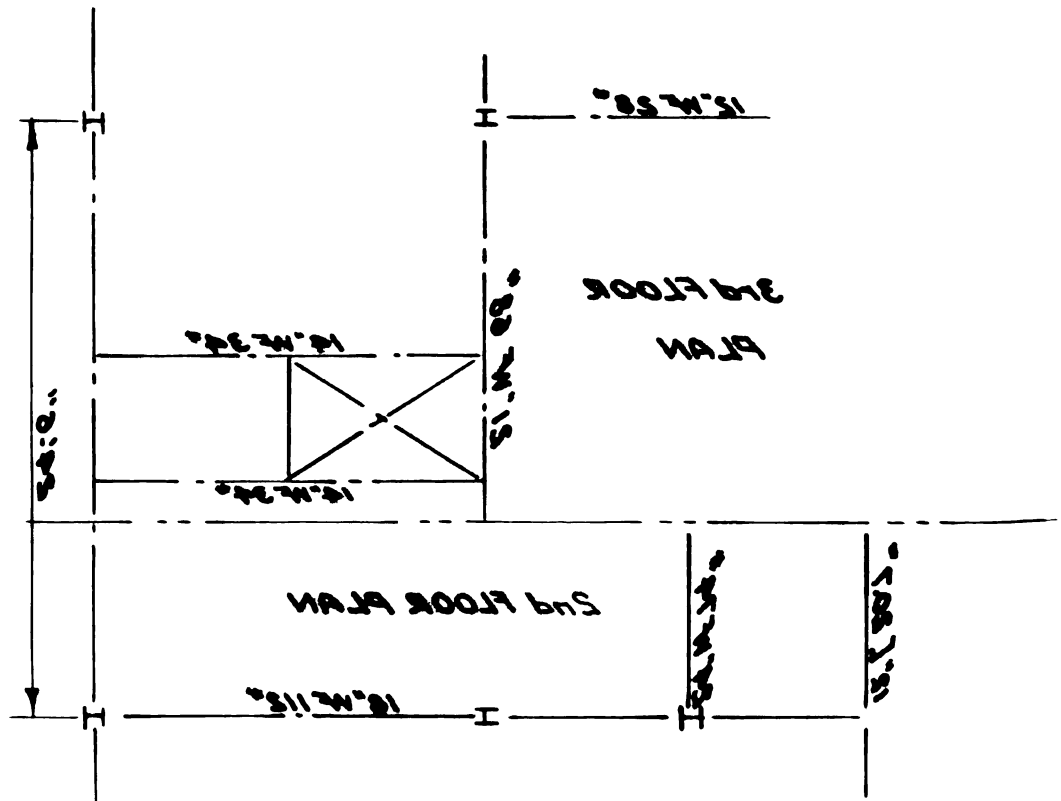


BALCONY DETAIL  
Scale -  $\frac{1}{8}" = 1'-0"$





BALCONY DETAIL  
Scale -  $\frac{1}{8}'' = 1'-0''$





The total load on the beam is  $1370 \text{ #/ft}$ . This load is applied over a length of  $18.7 \text{ ft}$ .

The total load on the beam is  $1370 \text{ #/ft}$ . This load is applied over a length of  $18.7 \text{ ft}$ . The total load is  $1370 \times 18.7 = 25,619 \text{ #}$ .

$$\left( \frac{1}{2} \times 1370 \times 18.7 \right) \div \frac{18.7}{2} = 25,619 \text{ #}$$

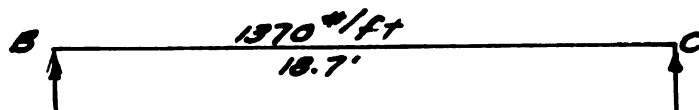
$$V = \frac{25,619 \text{ #}}{1.75 \times 12 \text{ in.}} = 1,201 \text{ #/in.}$$

The total load on the beam is  $1370 \text{ #/ft}$ . This load is applied over a length of  $18.7 \text{ ft}$ . The total load is  $1370 \times 18.7 = 25,619 \text{ #}$ .

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$$M = \frac{1}{2} \times 1370 \times 18.7 \times \frac{18.7}{2} = 25,619 \text{ #ft}$$

$$V = \frac{1}{2} \times 1370 \times 18.7 = 25,619 \text{ #}$$



The total load on the beam is  $1370 \text{ #/ft}$ . This load is applied over a length of  $18.7 \text{ ft}$ . The total load is  $1370 \times 18.7 = 25,619 \text{ #}$ .

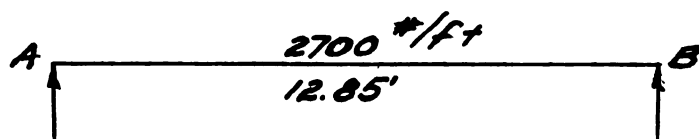
$$M = \frac{1}{2} \times 1370 \times (18.7)^2 = 25,619 \text{ #ft}$$

$$= \frac{25,619 \text{ #ft}}{12 \text{ in.}} = 2,135 \text{ #/in.}$$

$$V = \frac{1}{2} \times 1370 \times 18.7 = 25,619 \text{ #}$$

$$V = \frac{25,619 \text{ #}}{12 \text{ in.}} = 2,135 \text{ #/in.}$$

The total load on the beam is  $1370 \text{ #/ft}$ . This load is applied over a length of  $18.7 \text{ ft}$ . The total load is  $1370 \times 18.7 = 25,619 \text{ #}$ .



$$M_1 = \frac{12,800}{10.0} = 12,800 \text{ ft.}$$

$$\text{Total dead load} = (12,800 \times 10) = 128,000 \text{ lbs./ft.}$$

$$\text{Max. live load} = \frac{128,000 \times (11.0)^2}{8} = 17,920 \text{ ft./ft.}$$

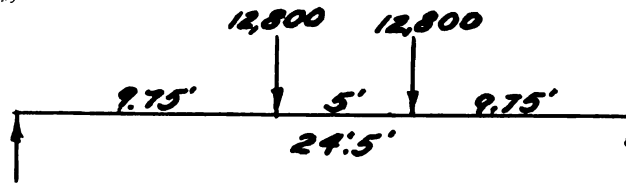
$$S = \frac{1,000 \times 11}{10.0} = 11,000 \text{ lbs./sq. ft.}$$

$$V = \frac{128,000 \times 11.0}{2} = 17,920 \text{ lbs.}$$

$$V = \frac{17,920}{10 \text{ ft.}} = 1,792 \text{ lbs./sq. ft.}$$

The live load is applied at right angles to the span sections and is 100 lbs./sq. ft.

$$\frac{128,000}{10} \times 10 = 128,000 \text{ lbs.}$$



$$\text{Total live load} = 12,800 (24.5 - 10.0) + \frac{12,800 (5.0)^2}{8}$$

$$= 128,000 + 4,000 = 132,000 \text{ ft./ft.}$$

$$S = \frac{132,000 \times 11}{10.0} = 14,520 \text{ lbs./sq. ft.}$$

$$V = \frac{12,800 + 4,000 \times 11.0}{11.0 \text{ ft.}} = 1,452 \text{ lbs./sq. ft.}$$

The live load is applied at right angles to the span sections:

Uniform load:

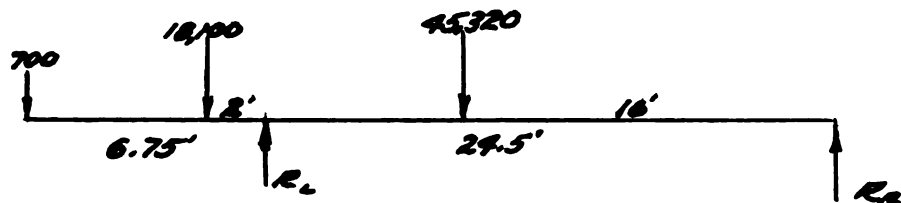
$$\frac{(128,000 + 4,000) \times 11}{2} = 10,700$$

$$\frac{10,700 \times 11}{11.0} = 1,070$$

$$\frac{11.0 \times 1,070}{2} = \text{Total} = 5,885 \text{ lbs.}$$

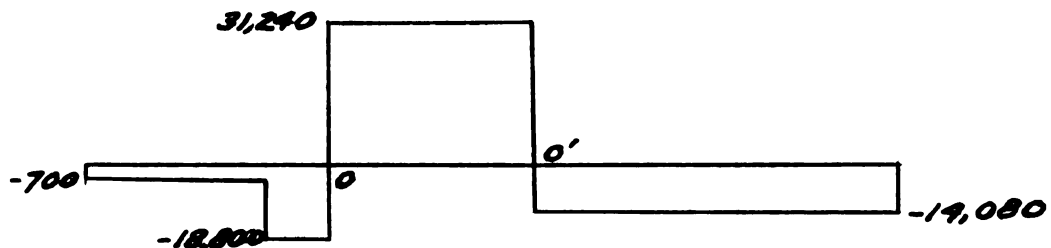
$$R_L = 700 + 18,100 + 45,320 = 64,120 \text{ lbs.}$$

2.



$$L = \frac{700 \times 31.75 + 18,100 \times 25 + 45,320 \times 13}{31.75} = 60,510 \text{ lbs.}$$

$$R = \frac{60,510 \times 31.75 - 18,100 \times 25 - 45,320 \times 13}{31.75} = 11,600 \text{ lbs.}$$



$$1. \text{ V. at } 0' = 11,600 \text{ lbs.} = 60,510 \text{ lbs./ft.}$$

$$2. \text{ V. at } 0' = 700 \times 31.75 + 18,100 \times 25 = 60,510 \text{ lbs./ft.}$$

$$3. \text{ } \frac{60,510 \times 31.75}{31.75} = 19,080 \text{ lbs.} \quad \text{ft.}$$

$$4. \text{ } \frac{11,600}{31.75} = 365 \text{ lbs./ft.}$$

5. The shear force diagram shows that the shear force is constant at -700 lbs from the left end to 6.75'. At 6.75', it jumps to +31,240 lbs. It remains constant at +31,240 lbs until 29.5'. At 29.5', it drops to -19,080 lbs. It remains constant at -19,080 lbs until the right end. The diagram crosses the zero line at 0' (the left end) and 0' (the right end).

6. The shear force diagram shows that the shear force is constant at -700 lbs from the left end to 6.75'. At 6.75', it jumps to +31,240 lbs. It remains constant at +31,240 lbs until 29.5'. At 29.5', it drops to -19,080 lbs. It remains constant at -19,080 lbs until the right end. The diagram crosses the zero line at 0' (the left end) and 0' (the right end).

$$V = 574 \quad L = 10.75 \quad R = 10$$

$$M = \frac{574 \times 10.75 + 10 \times 10}{10.75 + 10} = 1.11 \text{ lbs.}$$

$$L = \frac{2(1.11 \times 10 - 1)}{2(1.11 \times 10 - 1)} = 1.01 \text{ lbs.}$$

$$M = 10.75 - 1.01 = 9.74 \text{ lbs.}$$

$$D = 10.75 \times \frac{(10 + 1 + 1)}{10} = 11.75$$

$$1. \text{ } \frac{10.75}{10} = 1.075 \quad \text{ft.} = \frac{10.75}{10} \text{ ft./ft.}$$

$$1.1.1. \dots = \frac{2.1.1. \dots}{2} = 1.1.1. \dots$$

$$2.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$3.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$4.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$5.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$6.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$7.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$8.1.1. \dots = 1.1.1. \dots$$

$$9.1.1. \dots = \frac{2.1.1. \dots}{2} = 1.1.1. \dots$$

$$10.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$11.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$12.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$13.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$14.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

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$$16.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$17.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$18.1.1. \dots = 1.1.1. \dots + 1.1.1. \dots = 1.1.1. \dots$$

$$19.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$20.1.1. \dots = \frac{1.1.1. \dots}{2} = 1.1.1. \dots$$

$$21.1.1. \dots = 1.1.1. \dots$$

[illegible]

$$B = \frac{7}{10}, \quad C = \frac{1}{10}$$

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## VII

## REINFORCED CONCRETE COLUMNS-MAIN AUDITORIUM

The axially loaded columns will be the first to be investigated. They include all of the supports for the flat slab floor and also for the beam and slab construction on the west end and near the stage.

For an axially loaded column, the load which the column will carry is given by the formula:

$$N = f_c A [1 + (n-1) p]$$

$$\text{and } f_c = \frac{N}{A [1 + (n-1) p]}, \quad f_s = n f_c$$

Column No. 13 will be taken as a typical example. It is 26 in. x 30 in. cross section and has 8-1 in. sq. steel bars for compression. The column supports part of the lobby roof load, part of the fan platform load, the magnitude of which was taken from the engineer's estimate, part of the west main wall, and four intermediate floor loads.

## Loads:

|                                            |              |
|--------------------------------------------|--------------|
| Roof (80 + 30) (20 x 12) + 60 x 20         | 27,200       |
| Fan platform                               | 60,800       |
| Fourth floor (70 + 70) (20 x 12) + 50 x 20 | 34,600       |
| Third " 27,200 + 25,600                    | 52,800       |
| Second " 46,800                            | 46,800       |
| Wall 35 x 20 x 30                          | 21,000       |
| First floor slab                           | 125,000      |
| Dropped panel                              | 4,500        |
| Column Capital                             | 6,000        |
| Total load transmitted to column           | 378,500 lbs. |
| Weight of column                           | 7,300        |
| N                                          | 385,800 lbs. |

$$f_c = \frac{385,800}{780 [1 + (12-1) .0103]} = \frac{385,800}{888} = 445 \text{ p.s.i.}$$

$$f_s = n f_c = 12 \times 445 = 5,340 \text{ p.s.i.}$$

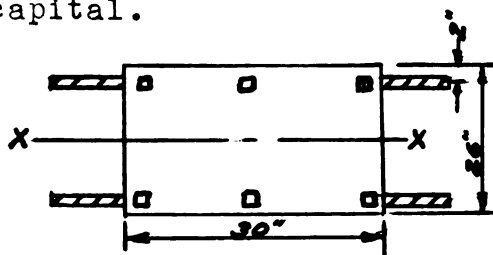
The allowable concrete stress in a short column with axial compression using a 2500 pound concrete is 565 p.s.i. It is necessary to determine whether the column is a short column.

## AXIALLY LOADED CONCRETE COLUMNS-MAIN AUDITORIUM

| Column | Size    | A   | Steel<br>Provided | As    | P     |
|--------|---------|-----|-------------------|-------|-------|
| 13     | 26 x 30 | 780 | 3-1" sq.          | 9.00  | .0103 |
| 14     | 30 x 30 | 900 | 3-1 1/8"sq.       | 10.13 | .0113 |
| 29     | 22 x 22 | 484 | 3-7/8" rd.        | 4.81  | .0099 |
| 30     | 22 x 22 | 484 | 3-7/8" rd.        | 4.31  | .0099 |
| 32 A   | 12 x 12 | 144 | 4-3/4" rd.        | 1.77  | .0123 |
| 35     | 24 x 24 | 576 | 11-1" rd.         | 8.64  | .0130 |
| 36     | 16 x 16 | 256 | 9-3/4" rd.        | 3.98  | .0155 |
| 45     | 22 x 22 | 484 | 11-1" rd.         | 8.64  | .0179 |
| 76     | 16 x 16 | 256 | 6-3/4" rd.        | 2.65  | .0133 |
| 83     | 16 x 16 | 256 | 4-7/8" rd.        | 2.41  | .0094 |

| Column | Load<br>Transmitted | Wt. of<br>Column | N       | fc  | fs    |
|--------|---------------------|------------------|---------|-----|-------|
| 13     | 378,560             | 7,500            | 585,860 | 445 | 5,340 |
| 14     | 384,600             | 8,450            | 593,050 | 388 | 4,660 |
| 29     | 244,360             | 4,530            | 248,890 | 463 | 5,570 |
| 30     | 339,690             | 4,550            | 344,220 | 640 | 7,690 |
| 32A    | 45,500              | 1,350            | 46,850  | 236 | 3,430 |
| 35     | 275,360             | 5,400            | 280,760 | 417 | 5,000 |
| 36     | 135,300             | 2,390            | 137,690 | 455 | 5,460 |
| 45     | 275,360             | 4,530            | 279,890 | 484 | 5,800 |
| 76     | 135,300             | 2,390            | 137,690 | 467 | 5,620 |
| 83     | 125,000             | 2,390            | 127,390 | 450 | 5,410 |

The maximum height of a short column is  $40 r$  where  $r$  is the least radius of gyration.  $H$  is taken up to the base of the column capital.



Moment of inertia about x-x:  
 Rectangle =  $\frac{30 \times (26)^3}{12} = 43,800$

Fins =  $3(7-1) \times \frac{(11)^3}{12} \times 2 = 7,986$   
 Total =  $51,786 \text{ in.}^4$

Area:

Rectangle -  $16 \times 30 = 480$   
 Fins -  $(7-1) \times 8 = 48$   
 Total =  $528 \text{ sq. in.}$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{51,786}{528}} = 9.55 \text{ in.}$$

Allowable  $h$  for a short column =  $40 \times 9.55 = 382 \text{ in.}$

Our length is  $144-35 = 109 \text{ in.}$  Therefore the column is in the short category and the compressive stresses are not excessive.

Besides the longitudinal reinforcements in this column are lateral ties. They are of  $\frac{1}{4}$ " rd. bars and spaced  $10"$  o.c.

A. C. I. also specifies that lateral ties be not less than  $\frac{1}{4}$ " in diameter nor spaced more than  $12"$  on centers. The design is within the limits.

A. C. I. also specifies that the longitudinal reinforcement be not less than .005 of the total area nor more than .02 of the area be used for  $p$  in calculations. For this column  $p = .0103$ .

The rest of the columns were inspected similarly and the results tabulated. In all cases the ties are spaced within the limits.

As a check on the allowable concrete stress, the column with the smallest dimensions and least steel will be tested.

The  $12" \times 12"$  column with  $4-\frac{3}{4}"$  rd. bars is the most likely one to be a long column.









It was necessary to conclude the investigation at this point due to lack of time. Practically all of the Main Auditorium was investigated, leaving the stage and part of the Theatre untouched.

However it is believed that all of the types of structural framework used in the building have been encountered. It can be said that further investigation would consist of repetitions of processes already completed. This does not mean however that further investigation is unwarranted but that the knowledge and information gained by the investigator would be far less in proportion to the work involved.

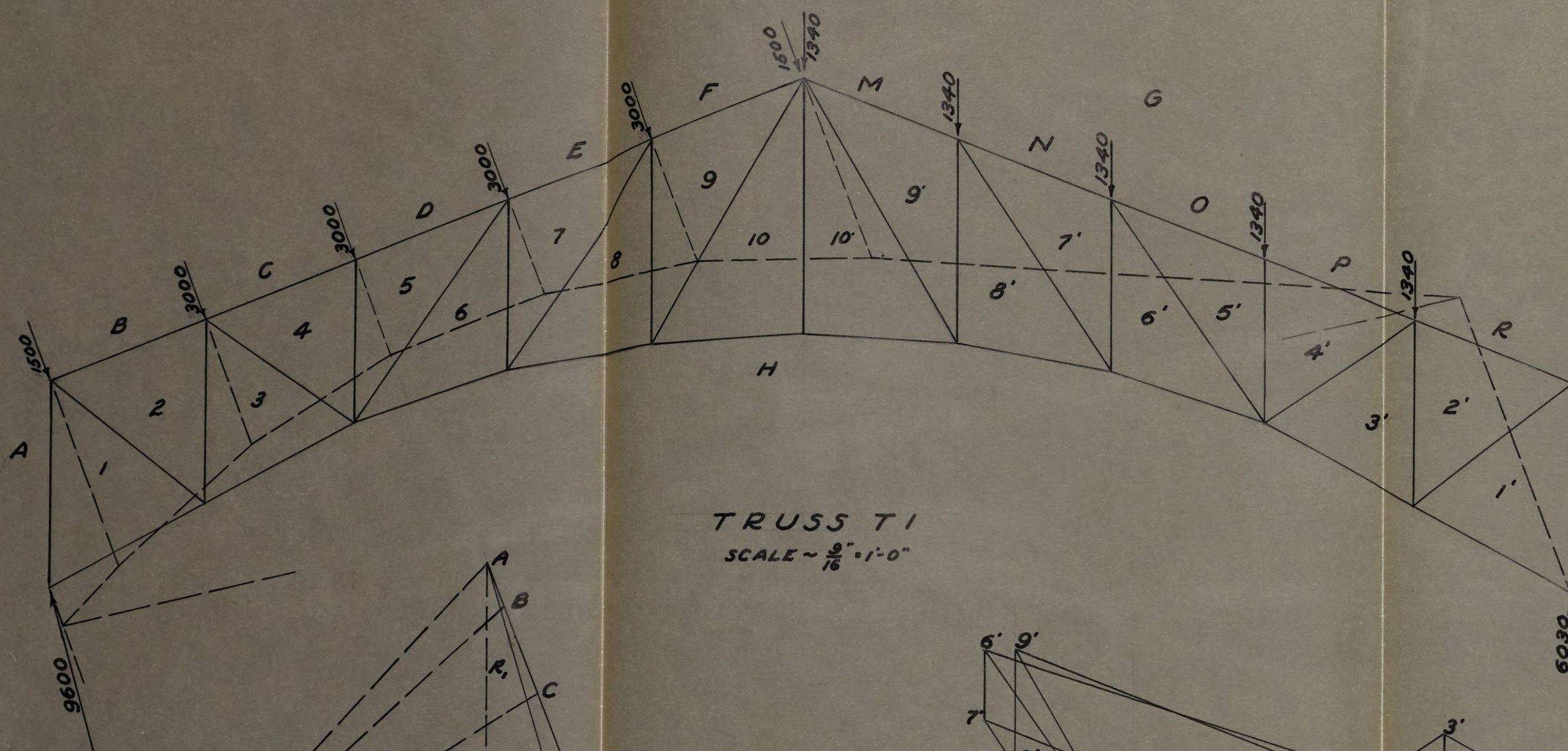
Many interesting features have been revealed during the process of the work and it is hoped that the reader has gleaned some of the different things involved in an unusual structure of this type.

~~SECRET~~

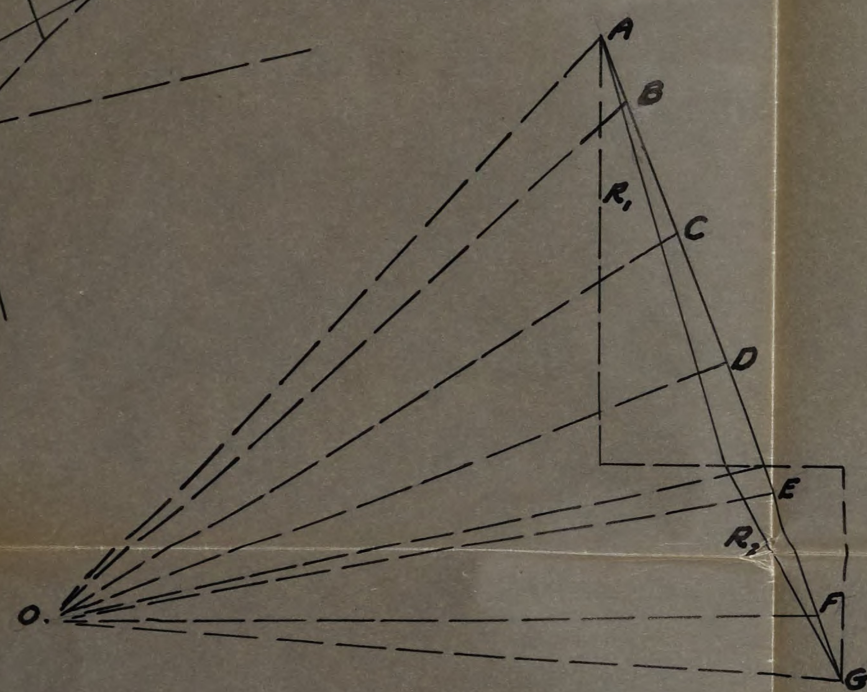
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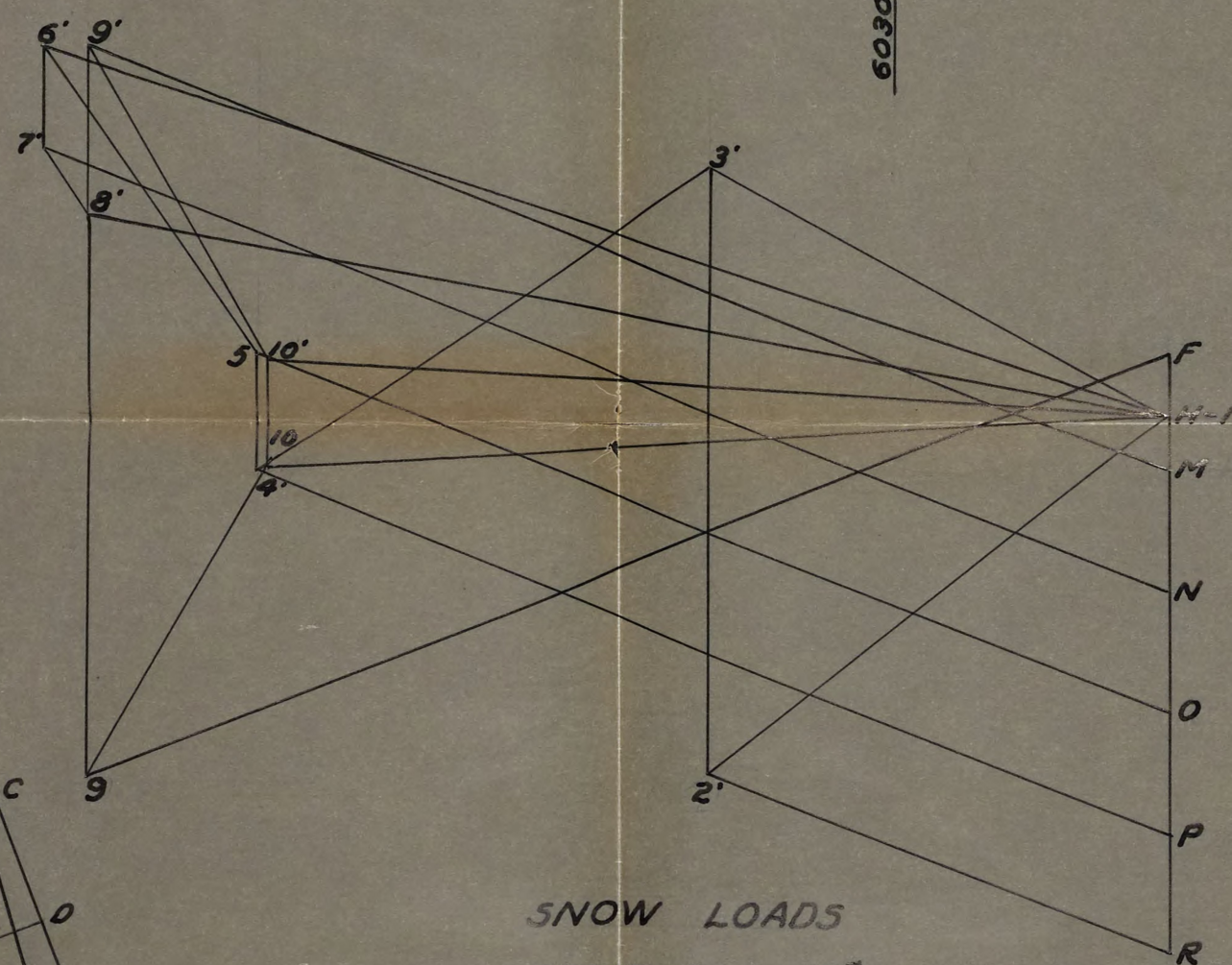




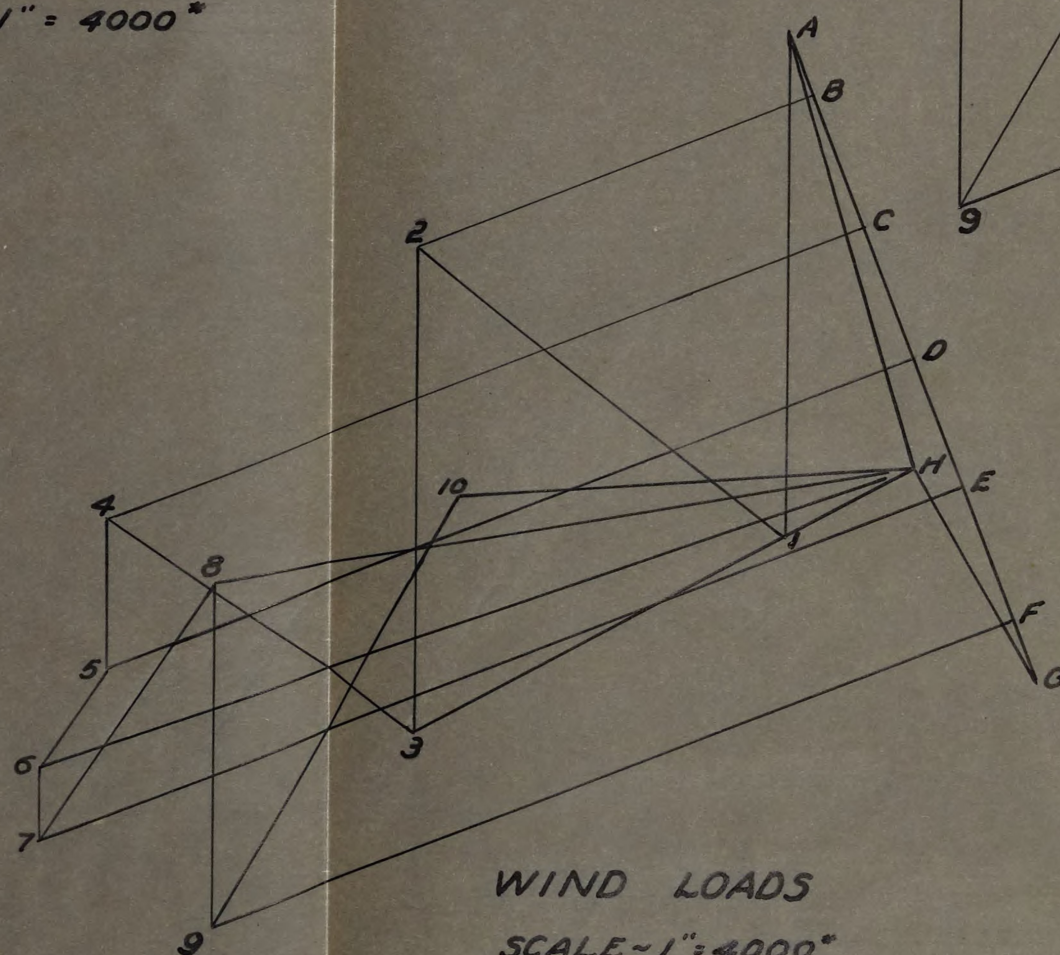
TRUSS T1  
SCALE ~  $\frac{9}{16}$ " = 1'-0"



SCALE ~ 1" = 4000'



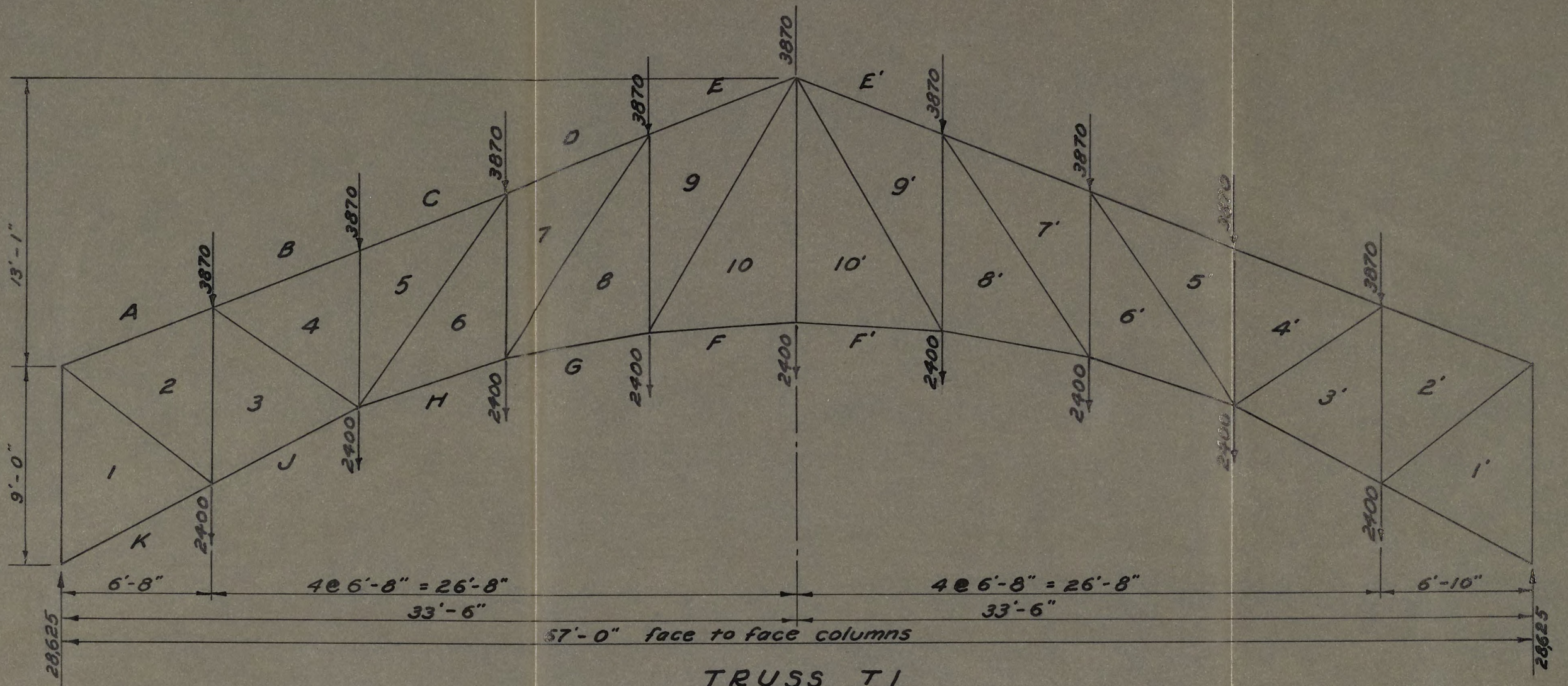
SNOW LOADS  
SCALE ~ 1" = 2000'



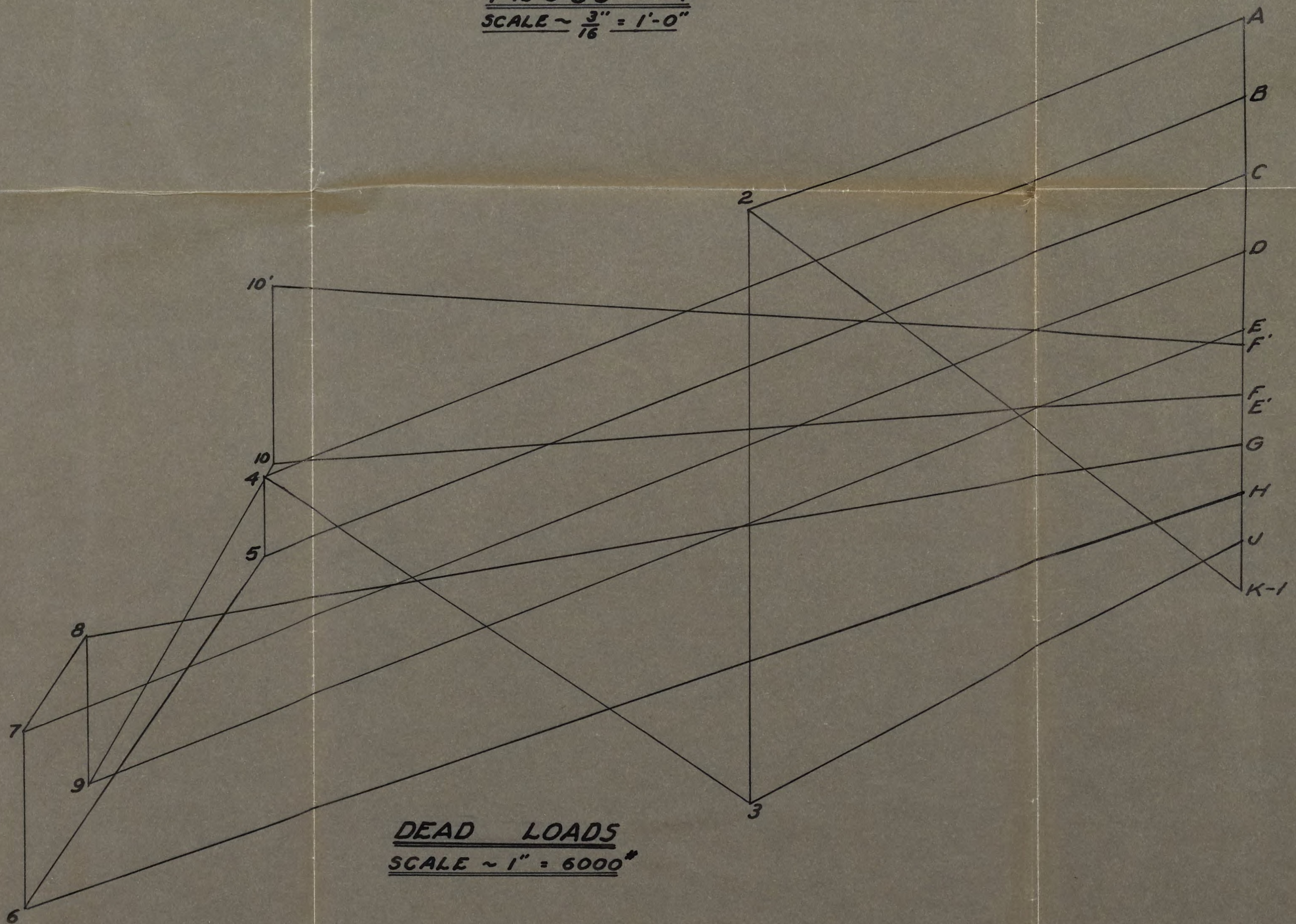
WIND LOADS  
SCALE ~ 1" = 4000'

MICHIGAN STATE COLLEGE  
GRAPHICAL STRESS ANALYSIS  
SCALE AS SHOWN DWG NO 1  
5-20-39 L. E. Townsend





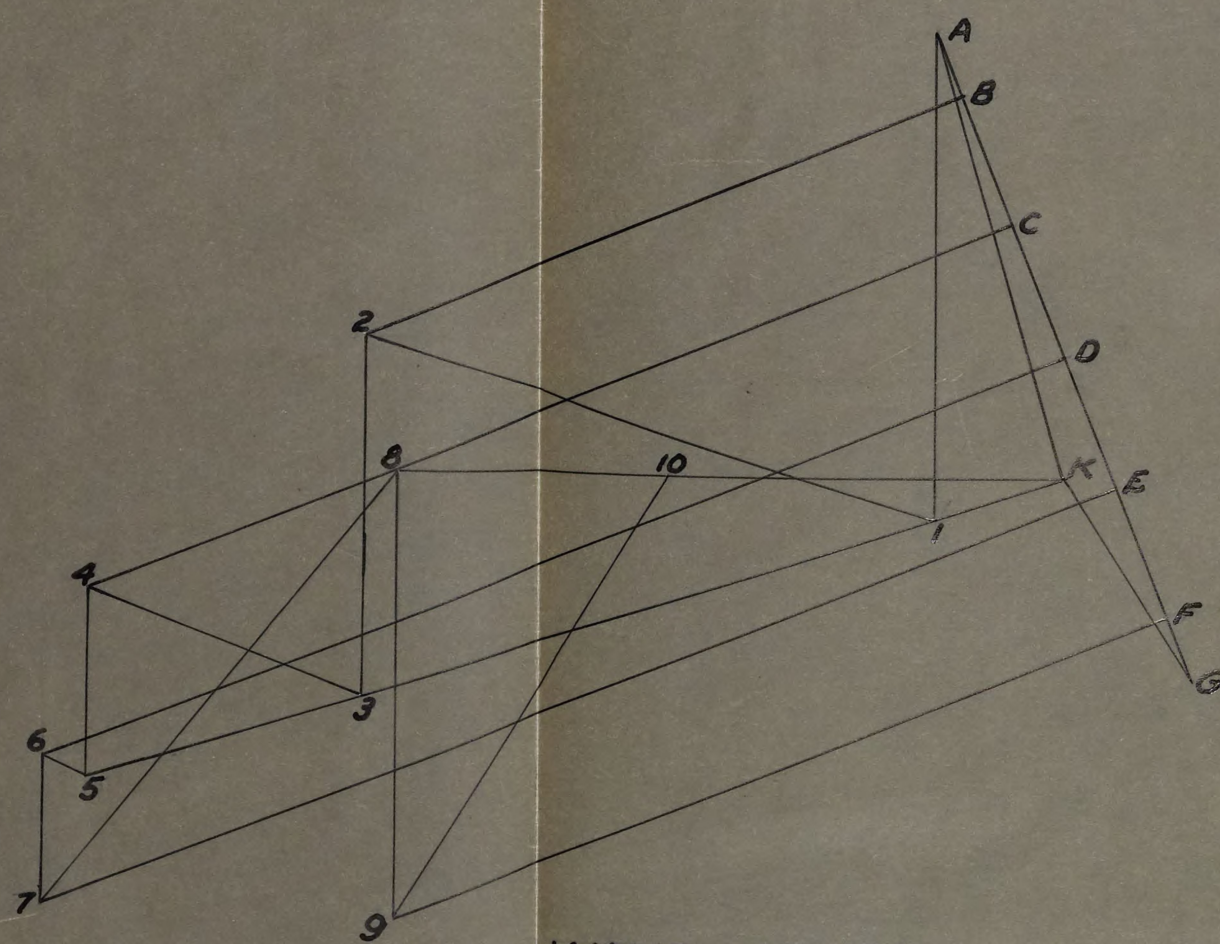
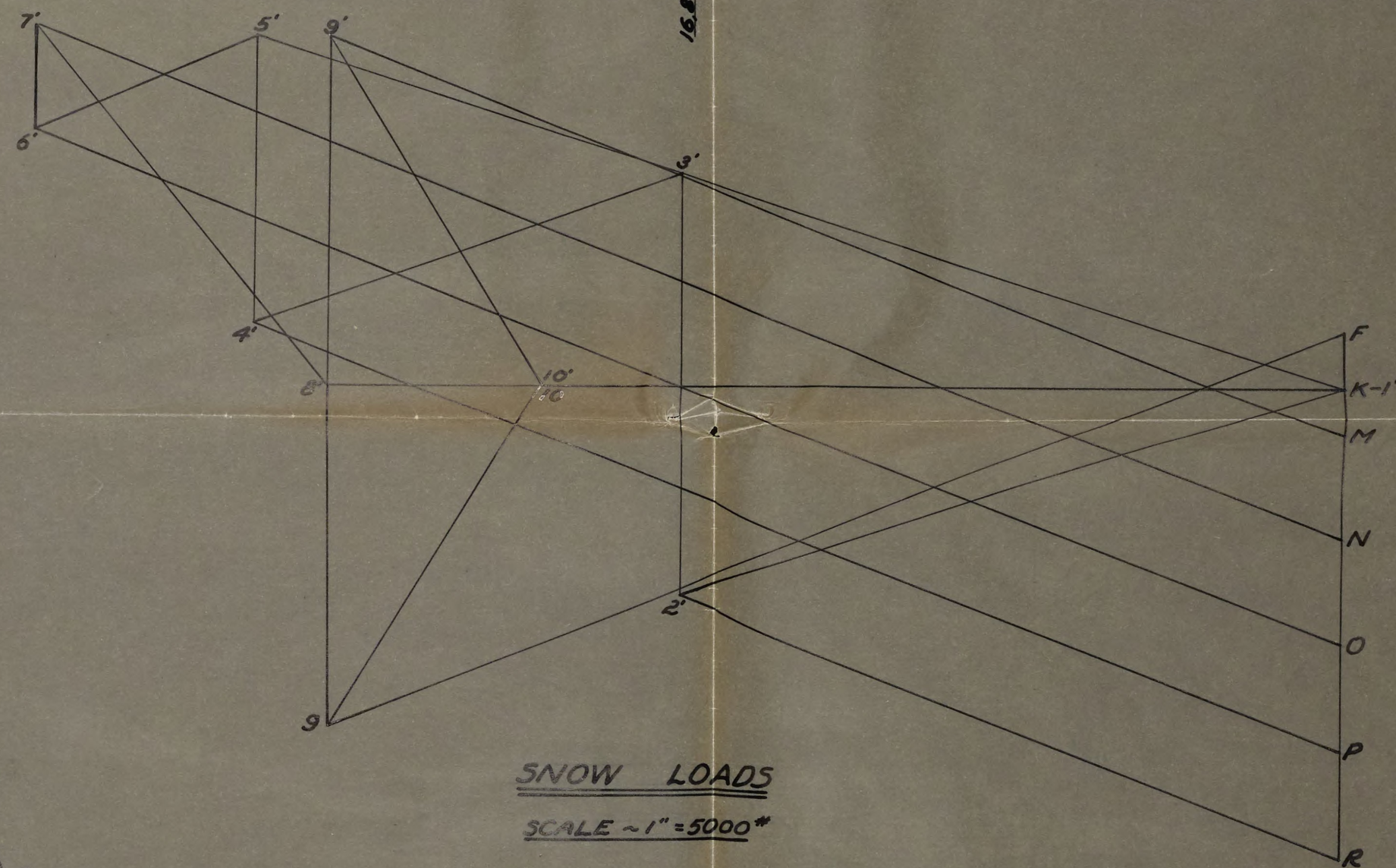
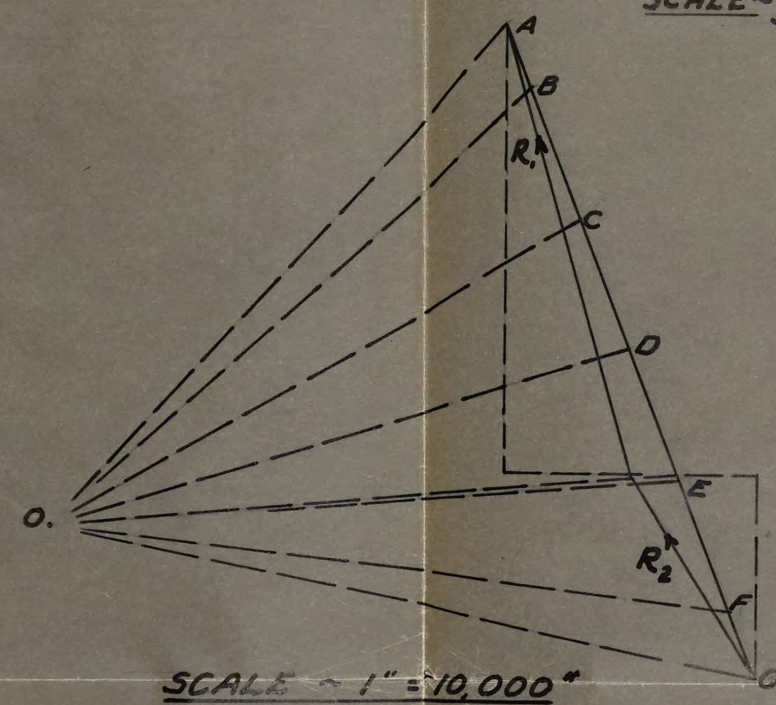
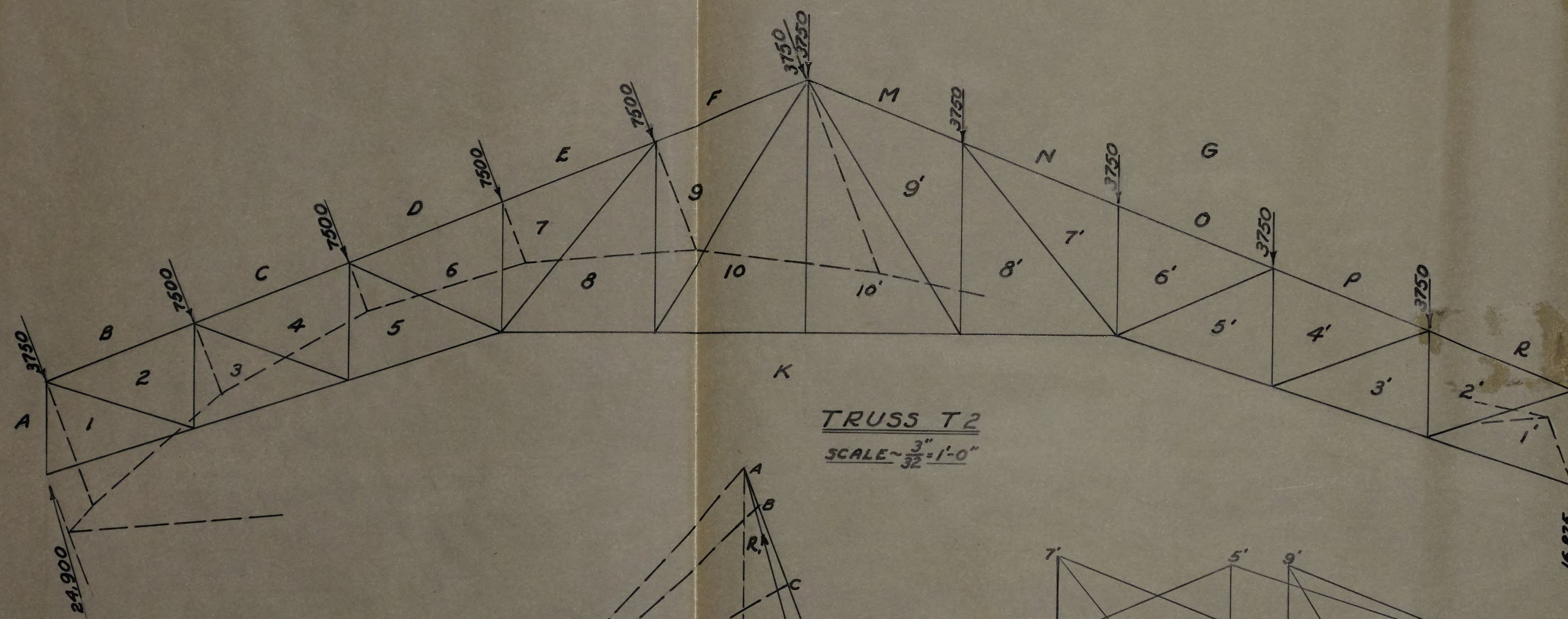
**TRUSS T1**  
 SCALE ~  $\frac{3}{16} = 1'-0"$



**DEAD LOADS**  
 SCALE ~ 1" = 6000\*

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 GRAPHICAL STRESS ANALYSIS  
 SCALE AS SHOWN DWG No 2  
 5-20-39  
 L.E. Townsend



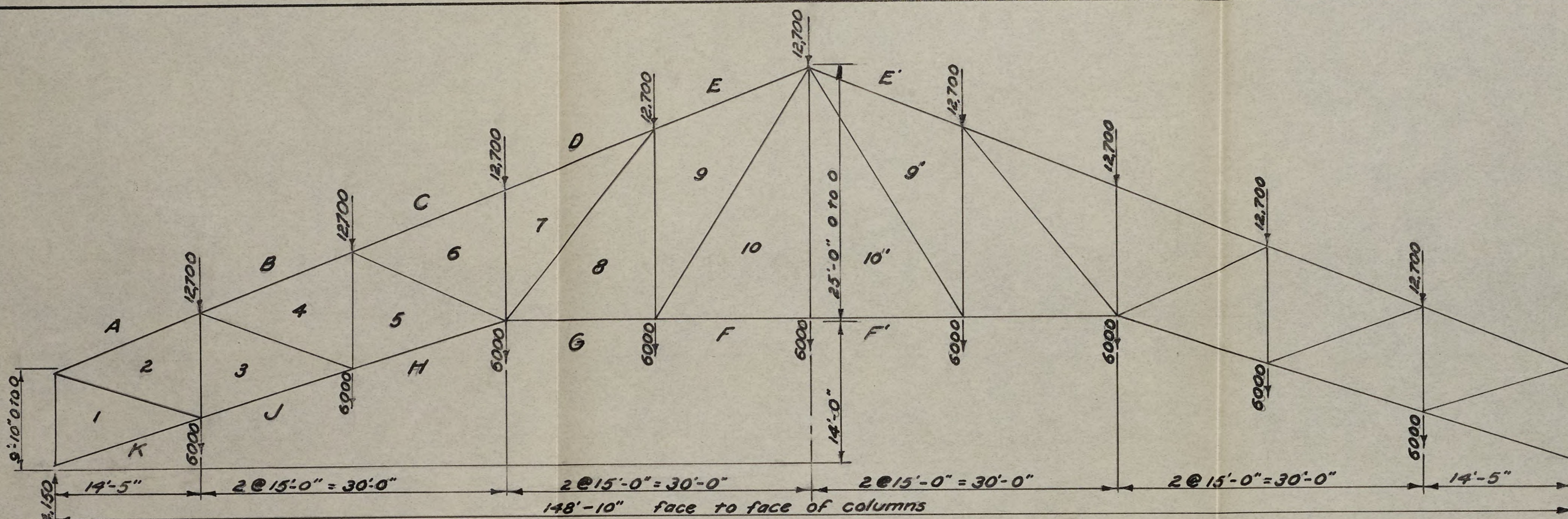


WIND LOADS  
SCALE ~ 1" = 10,000 #

SNOW LOADS  
SCALE ~ 1" = 5000 #

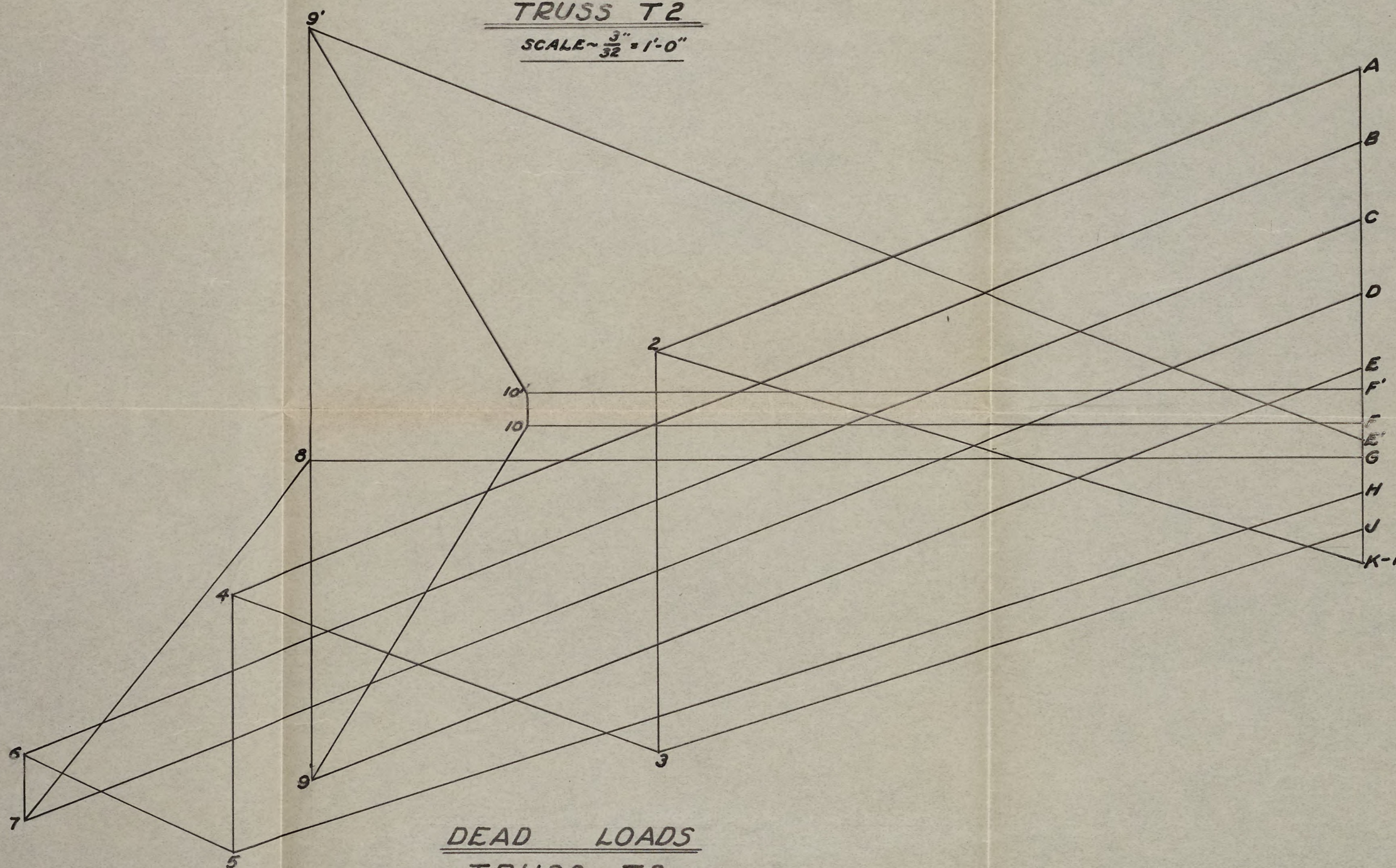
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GRAPHICAL STRESS ANALYSIS  
SCALE AS SHOWN DWG NO 3  
5-20-39 L. E. Townsend





TRUSS T2

SCALE ~  $\frac{3}{32}$ " = 1'-0"



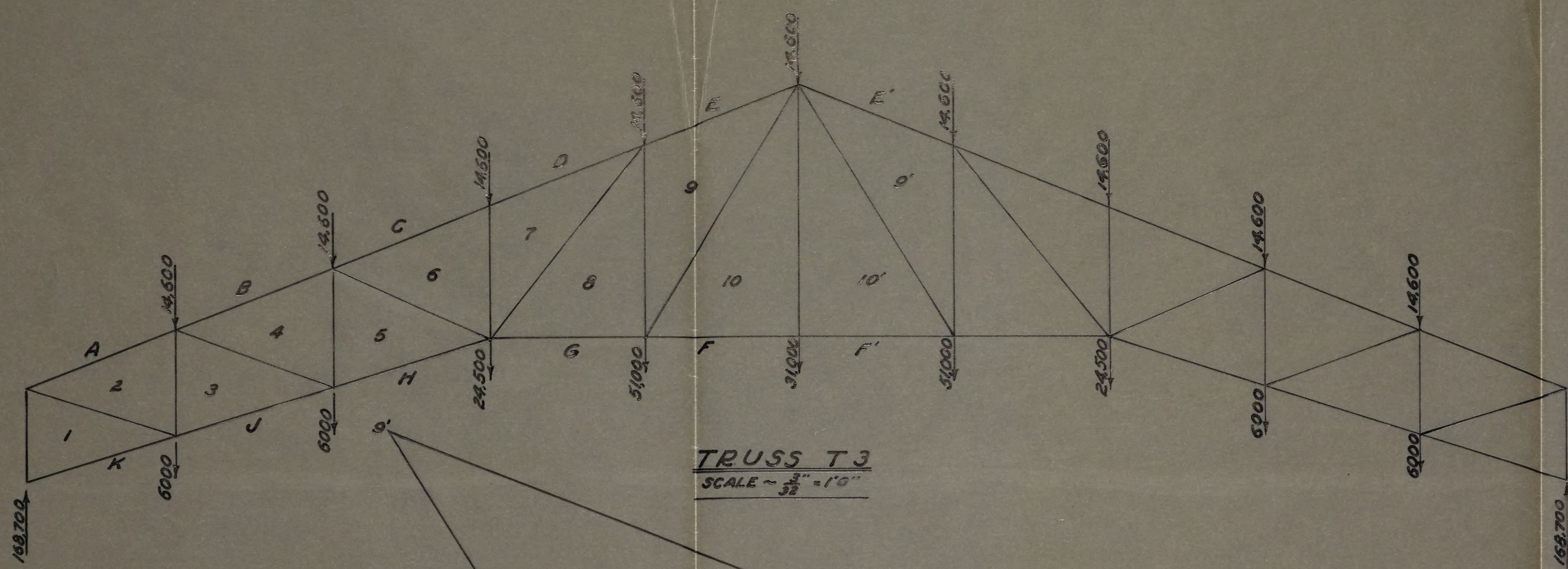
DEAD LOADS

TRUSS T2

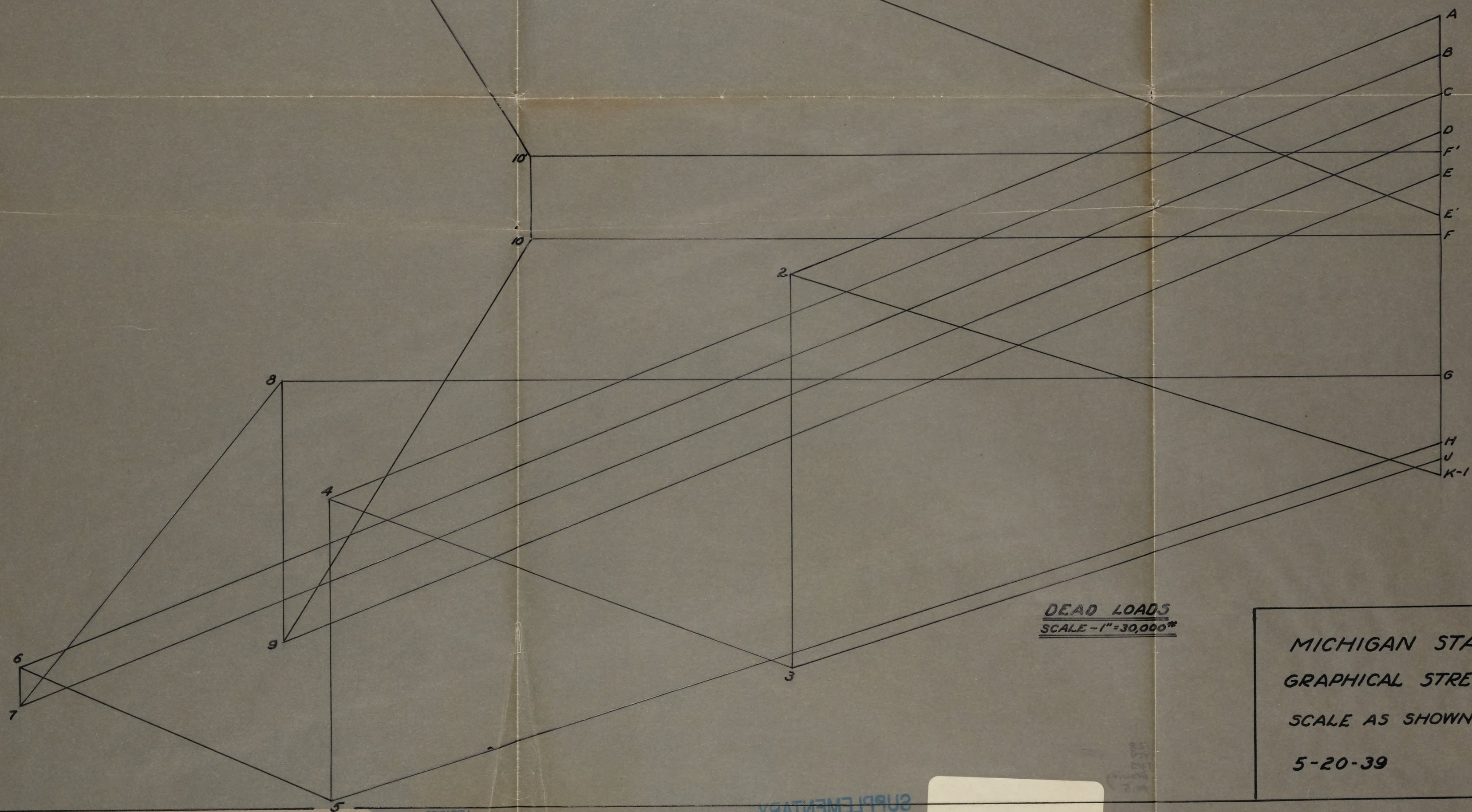
SCALE ~ 1" = 20,000#

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 GRAPHICAL STRESS ANALYSIS  
 SCALE AS SHOWN DWG NO 4  
 5-20-39 L. E. Townsend





TRUSS T3  
SCALE -  $\frac{1}{32}'' = 1'-0''$



DEAD LOADS  
SCALE - 1" = 30,000#

MICHIGAN STATE COLLEGE  
GRAPHICAL STRESS ANALYSIS  
SCALE AS SHOWN DWG N° 5  
5-20-39 L. E. Townsend





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