Design of a Steel Head-Frame

Civil Engineering

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DESIGN

OF A

STEEL HEAD-FRAME

BY

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THESIS

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This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

DJALMA DOWNEY WILLIAMS

entitled DESIGN OF A STEEL HEAD-FRAME FOR A MINE

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Head of Department of Civil Engineering.
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ART. I. INTRODUCTION.

"The subject of this thesis, "The Design of a Steel Head Frame" is one of the new problems that have been presented to structural engineers within the last few years. It has been but a very short time since all headframes were constructed entirely of wood. At that time, there was reason for this, such as abundance of suitable timber, ease of construction, and the reluctance of mine owners to depart from the practice of the day.

However, with the growing scarcity of timber, and with the
great advances made in engineering, it was seen that steel for this kind of construction had many advantages over timber, and so it was accordingly adopted.

The purpose of the writer is not to make a design for any particular mine, but to make a design for a mine having a daily output of from 400 to 600 tons, the depth being about 500 feet.
ART. 3. SPECIFICATION.

This steel-head frame is to be designed according to Theodore Cooper's General Specifications for Steel Highway and Electric Railway Bridges and Vessels, revised edition of 1901; with the following modifications.

**General.**

The head frame is to be designed for a shaft 500 ft deep having double compartments, each compartment being 4 ft × 4 ft in the clear, with 12-inch timber walls, and partition.

**Dimensions.**

The height of the head frame is
to be 60 ft from top of foundation to the centre of the sheaves. The
base shall be 30 ft by 32 ft measured upon the foundations from the centre
of the main posts.

Cable:

A 1½ in. wrought iron steel cable having
an ultimate stress of 116,000 lb, and with
an allowable working stress of
24,000 lb, is to be used.

Weight of carriage, car and cable.

Weight of car is 2,000 lbs when
empty, and 6,000 lbs when loaded.
Weight of hoisting carriage is 4,000 lb
and weight of the cable is 2,000
lb.

It is obvious that the maximum
weight of 12,000 lbs comes on the cable.
when a loaded car is starting from the bottom of the shaft. Allowing a factor of safety of two to account for stresses due to changes in acceleration, as for instance those stresses due to starting, stopping, and wedging, this gives an allowable working stress of 24000 lb which agrees with the allowable working stress of 24000 lb for a 1/4 inch cable as given above.

Sheaves.

There are to be two sheaves, each nine feet in diameter. The sheaves are to have cast iron rims and wrought iron spokes, and each is to be designed as to safely bear the breaking stress of the cable.
Dead Load.

The dead load is to be taken as the dead weight of the structure.

Live Load.

The live load is to be taken as a breaking stress on one cable and a maximum working stress with a factor of safety 2 upon the other cable, i.e., \(116,000 \text{ lb} + 24,000 \text{ lb} = 140,000 \text{ lb} = 70 \text{ tons.}\)

Wind Load.

The wind load is to be taken at 30 lb per sq ft of vertical projection, and is to be considered on all four sides in succession.

The wind bracing shall be so designed as to take both tension and compression and all connections are to be riveted.

In all stresses, the plus sign \((+)\) will be used to denote tension, and the minus sign \((-)\) to denote compression.
DETERMINATION of RESULTANT under Maximum Loading.

Ground Plan. Fig 3.
ART. 3. DETERMINATION OF STRESSES.

A.-Wind Stress.

A wind force of 30 lbs per sq ft is considered acting in succession on the right and left of the headframe, as shown in Fig 3 Plate II.

For the first, or bottom panel, there is a wind force of 11,800 lbs; for the second panel the wind force is 8,400 lbs; for the third panel the wind force is 5,000 lbs. In each of these three panels the load is considered as being equally divided among the four panel points. In the fourth or top panel the wind force is 18,400 lbs, one half of this is applied at the two lower panel points and the remaining half is applied at the top of the panel. The determination of the wind stresses in the various
PLATE, VI.

Axle of Sheave. Fig. 1.

\[ \text{Me} = 4.75 \times S_2 - 24 \times 2\frac{1}{2} = 0 \]
\[ S_2 = 12.6 \]
\[ M_d = 9.5 \times S_1 + S_1 \times 4.75 - 116 \times 2\frac{1}{4} \]
\[ -24 \times 7\frac{1}{4} = 0 \]
\[ S_1 = 66.0 \]
\[ S + S_1 + S_2 - 116 - 24 = 0 \]
\[ S = 61.4. \]

Horizontal thrust is
\[ = 6300 - 1300 = 5000 \text{#} \]

DETERMINATION of the LIVE LOAD CONCENTRATION.
$M_a = 5 \times 5000$
$= 25000 \text{ lb ft.}$

$M_b = 5000 \times 8$
$= 40000 \text{ lb ft.}$

**STRESS DIAGRAM**

for

**CENTRE VERTICAL LOAD**

**AND HORIZONTAL THRUST.**

**ON BACK BENT.**

0. 1. 2. 3. 5000 lbs.
B. LIVE LOAD STRESSES.

The live load is considered as acting through the axle of the sheaves at the three points of bearing; also as acting in the direction of the resultant of the maximum stress, due to the pull on the cable.

The axle of the sheaves under a breaking stress in one cable, and a working stress in the other would act as a continuous beam, but in account of the difficulties of the construction of movable bearings, and also to be on the safe side, it will be considered as two simple beams.

The unequal loading on the sheave creates a horizontal thrust which causes stresses in the diagonal bracing of the rear bent.

For the determination of the live
C DEAD LOAD STRESSES.

The dead load will for the most part come on the six main posts. The lateral bracing will be considered as having no appreciable dead load.

In determining the dead load a preliminary design will be made. This will be done by considering the average stress in the rear main posts, due to the live and wind load only.

Sections same as obtained for the rear posts, will be used for those in front. This is done in order to provide for the effect of the great vibrations, which occur in a structure of this character, and also to provide for the stresses due to a coal tipple, a breaker, or any other structure that may in the future be connected to the headframe.
ART. 4. PRELIMINARY DESIGN.

MAIN POSTS.

The main posts will be composed of a section, consisting of four Z-bars and one web plate. This kind of a column is considered to be of such a character best suited to the conditions as, in account of its large radius of gyration per square inch of section, it can economically withstand the effect of the vibrations, and also it is very accessible for efficient painting.

It is required to design the main post in order to determine the dead load. The average stress in the post due
to withstand and bear loads is 110,000 pounds. Try an 8-in. Z-bar column, composed of four Z-bars 4" x 3/8" and one web plate 7" x 3/8". The length is 16 feet. The least radius of gyration, ρ, is 2.57 inches, the area 17.1 sq in. and the weight 58 pounds per linear foot.

Taking the radius of gyration as 2.57 inches, the value of \( \frac{1}{\rho} \) equals \( 16 \times \frac{12}{2.57} = 74.5 \). \( P \) equals 6650 pounds per square inch, and the required sectional area, 16.5 sq in., is somewhat less than the area of the column chosen, 17.1 sq in.

This section is sufficiently close, as great nicety is not required in preliminary design, and will be used to determine
The dead load.

To allow for the weight of connections, rivets and gusset plates, 20% will be added, thus making the total weight of a column equal to:

$$928 + 0.20 \times 928 = 1115 \text{ pounds}.$$
MAXIMUM STRESSES.
ART. 5. FINAL DESIGN.

A. Main Post Section.

For the three bottom sections of the main posts, the same section will be used as is required for the lower one.

The total stress to be considered is 12,1300 pounds, the length is 16 feet. The post will consist of a section composed of four Z-bars 4" x 7/8" and one web plate 9/16". The least radius of gyration is 2.49 inches, and the area 19.0 sq. in. The value of \( \frac{F}{A} \) equals \( \frac{16 \times 12}{2.49} = 77.1 \) which gives a value of 6,530 pounds for the allowable unit stress. The required sectional area is therefore \( \frac{12,1300}{6,330} = 18.6 \) sq. in. As the assumed sectional area is
slightly larger than the required area, this section is considered as being sufficiently strong.

As previously mentioned, the front posts are considered as having the same stress as the rear posts. The section given above will be used for all of the main posts.

B - Lateral Bracing.

The lateral bracing between the rear posts, will be considered starting from the bottom and going towards the top in succession. The first member has a stress of 11,950 pounds in compression and a length of 18.9 feet. A section consisting of four 4" x 3" x 3/16" angles placed in pairs with their short legs back to back, with a 1/8 inch lacing, will
be used. Back to back of angles is 10 inches. The least radius of gyration 1.79 inches, gives a value of 118 \( \sqrt{ } \). The allowable unit stress is 5900 pounds, and this value divided into the total stress gives a required sectional area of 3.10 sq ins.

This section will also be used for the second horizontal members.

The stress to be considered in the third horizontal member is 14000 pounds in tension. A trial shows that a 4\( \times \)4\( \frac{1}{2} \) angle is sufficiently large, when allowing one leg as being effective.

All other diagonal bracing being in tension, a trial shows that 4\( \times \)4\( \frac{1}{2} \) angle is sufficiently strong in each case.

- C-SIDE LATERAL BRACING -

The maximum stress in the horizontal members is 7600 pounds in compression, length 22 feet 5 inches.

By a trial it is shown that the section
used for the horizontal members between the rear posts, is sufficient in this case.

In the diagonal members there is a maximum tension stress of 8400 pounds. A single 4" x 4" x 3/8" angle will be used for each member, and each pair will be riveted at their place of intersection. The same sections will be used for the front lateral bracing, as was used for the rear bracing.

Guides and Bracing.
The stresses in the guides and bracing are rather indeterminate. However, we know that the wind stress, and stresses due to the friction of the carriages, will be taken up by these members.
The design is such that practice and experience has shown it to be sufficient for all practical purposes.
D-SHEAVE.

The sheave having sixteen spokes is designed similar to a bicycle wheel, leaving tensile spokes only, as explained in Murrin's and Jacoby's "Rope and Bridges" Part III. The sheave diagram on Plate IX gives a tensile stress of 55,000 pounds in each spoke. The area of cross-section of a steel rod to take this stress is 0.5 sq in. Rods 1 in. in diameter will be used. The rim is designed as a simple beam uniformly loaded with 116,000 pounds between two adjacent spokes. From $M = \frac{SI}{c}$ letting $S = 8000$ pounds per sq in., the area required is 10 sq inches.

E-AXLE.

The axle is treated as a simple beam, having a distance of 5 feet between supports. Using a unit moment, there of 15,000 pounds per sq in. gives a required diameter of 5 inches.
It is connected to the headframe.
## ESTIMATE OF WEIGHT

<table>
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<tr>
<th>Ref No</th>
<th>NAME OF PIECE</th>
<th>No</th>
<th>SIZE (inches)</th>
<th>Length (in feet)</th>
<th>Wt in lbs per lin ft</th>
<th>Weight per piece</th>
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<tr>
<td>1</td>
<td>Z-Bar Columns</td>
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<td>64.7</td>
<td>8500</td>
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<td>2</td>
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<td>8&quot;</td>
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<td>7500</td>
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<tr>
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<td></td>
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20% For connections. = 7210  
4% For rivets. = 1440  

Total Weight = 8650  

---

Total Weight = 44710
**ESTIMATE OF COST.**

Cost of Materials:
- 44700# @ 1.84 cts  \[ \text{\$822.60} \]

Shop Cost:
- 22.3 Tons @ \$15.00  \[ \text{\$334.50} \]

Erection Cost:
- 22.3 Tons @ \$13.00  \[ \text{\$289.90} \]

Drafting
- 22.3 Tons @ \$4.00  \[ \text{\$89.20} \]

Hauling:
- 22.3 Tons 25 mi @ 25 cts per ton mile  \[ \text{\$139.38} \]

Painting:
- 22.3 Tons @ \$1.00 per ton  \[ \text{\$22.30} \]

Total = \$1697.86

\[
\text{\$1697.86} \div 44710 = 3.8 \text{ \$ per lb.}
\]

= Average pound price.