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TECHNICAL BULLETIN No. 13
OREGON STATE HIGHWAY DEPARTMENT
R. H. BALDOCK
Chief Engineer

Rational Design Methods for Short-span Suspension Bridges for Modern Highway Loadings

By

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and
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For City Council of Portland 10 Nov-10
Submitted by
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OREGON STATE HIGHWAY COMMISSION
SALEM
HENRY F. CABELL, Chairman
HURON W. CLOUGH, Commissioner
HERMAN OLIVER, Commissioner

(also 2 Photos)

July, 1940

FOREWORD

In a former publication by this department* certain economic data pertinent to short-span suspension bridges were presented, the findings, in general, indicating that the suspension type, in many instances, constitutes a most effective and economic selection for short-span structures. In this connection, the writers wish to point out a few fundamental facts:

Suspension bridge designs, classified as to the method of stiffening the suspension system may be segregated into three groups: (1) unstiffened, (2) integrally stiffened, and (3) deck stiffened.

Structures in the first classification are not suitable for short-span highway bridges due to the fact that such structures are lacking in rigidity, and that the live loads produce excessive distortions, thus rendering the designs unsuitable for traffic.

Structures in the second class are suitable for highway traffic; however, this type has not enjoyed extensive use due principally to the excessive cost.

The deck-stiffened type has been used extensively, and has proved satisfactory. However, it possesses certain peculiarities which may at first mislead the designer when proportioning its various parts. This type comprises, as it were, a dual ensemble —a primary and a secondary part. The primary component is the suspension system (cables, hangers, and deck). Its function is to support the loads. It is essential to stability. The secondary component is the stiffening frame. Its purpose is to reduce those live-load distortions which, if not thus limited, would produce grade changes of a magnitude sufficient to render the structure unsuitable for heavy or high-speed highway traffic. The stiffening truss, in addition to its principal function, also operates to carry a certain proportion of the live

* Oregon Highway Department Technical Bulletin No. 11.

load directly to the supports by virtue of its simple beam capacity. Since these two systems act simultaneously it is obvious that the stress distribution in the ensemble will not be determinate from statics alone.

Stiffening frames may be either hinged or fixed at the towers, and the side spans may be either suspended or independently supported. Since space will not permit a consideration of all of these types, the present investigation has been limited to structures with *suspended side spans* and *two-hinged* stiffening frames.

As may be inferred from the title, this bulletin will be confined to suspension bridges of comparatively short-span (from 350 feet to 600 feet, main span length). In a subsequent bulletin under process of preparation at this time, the researches will be extended to longer spans, and an attempt made to develop certain design constants of universal application.

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$L_E = l\{\frac{1}{4}[5/2+16\phi^2] \times [1+16\phi^2]^{1/2} + 3/32\phi \ln[4\phi + (1+16\phi^2)^{1/2}]\}$; $\phi = f/l$ = sag ratio

order to simplify the analysis, the effect of the side spans will be neglected for the present, and the cable structure considered as fixed at the tower points "O" and "D".*

The external loads acting on the cable consist of the dead load of the entire system (cables, stiffening frames, deck, etc.) which may be represented by the term "w" per unit length and the *effect* of the live loading which may be represented by the term "q" per unit length. The dead load "w" may be considered as uniform throughout the length of the span within the limits of accuracy desired. The live load "q" *may or may not* be considered as uniformly distributed, depending upon the degree of accuracy desired (this will be discussed later). It is, in fact, variable, as will be seen from Equation 8. In this connection it should be pointed out that the loading "q" does not represent the *total live load* but only *that portion* of the live load transmitted to the cable.

Let S_D represent the dead-load stress in the cable at any point;
S_L represent the live-load stress in the cable at any point;
λ represent the unit distortion of the cable under live load;

Then the average stress during the period of motion was $S_D + \frac{1}{2}S_L$ and the total resilient energy stored within the cable is

$$W_I = \int_0^l [S_D + \tfrac{1}{2}S_L]\, d\lambda \quad \ldots\ldots (1)$$

But, from Figure 2,

$$\frac{d\lambda}{d\Delta} = \frac{S_L}{AE}$$

$$S_D = H_D \cdot \frac{ds}{dx} \text{ and } S_L = H_L \cdot \frac{ds}{dx}$$

$$\text{Also } d\lambda = S_L \frac{ds}{AE_c} = H_L \cdot \frac{ds}{dx} \cdot \frac{ds}{AE_c}$$

* Side-span effects will be considered later.

Where:

H_D = Horizontal stress in cable due to dead load;
H_L = Horizontal stress in cable due to live load;

$$\lambda = \frac{S_L}{AE_c} \qquad \lambda AE_o = S_L$$

Whence

$$W_I = [H_D + \tfrac{1}{2}H_L]\left(\frac{H_L}{AE_c}\right) \cdot \int_0^l \left(\frac{ds}{dx}\right)^2 ds \quad \ldots\ldots (2)$$

$$= [H_D + \tfrac{1}{2}H_L] \cdot \left(\frac{H_L}{AE_c}\right) \cdot L_E\,^*$$

Expressed in words, the internal energy stored within the cable equals the product of the dead load horizontal pull, plus one-half the live load horizontal pull, multiplied by the live load horizontal pull times the length integral

$$L_E = \int_0^l \left(\frac{ds}{dx}\right)^2 ds$$

divided by the area of the cable, times its modulus of elasticity.

Reasoning in a similar manner, the average external loading during the period of motion is represented by the term $(w + \frac{1}{2}q)$; hence the external work which has generated the resilient energy represented by Equation 2 (and therefore must balance it) is represented by the expression

$$W_E = \int_0^l (w + \tfrac{1}{2}q) \cdot \Delta \cdot dx \quad \ldots\ldots (3)$$

Where Δ represents the deflection of the cable at any point.

For elastic equilibrium, therefore

$$[H_D + \tfrac{1}{2}H_L]\left(\frac{H_L}{AE_c}\right)[L_E] = \int_0^l (w + \tfrac{1}{2}q) \cdot \Delta \cdot dx \quad \ldots\ldots (4)$$

* The term L_E is the integral $\int_0^l \left[\frac{ds}{dx}\right]^2 ds$ which can be readily evaluated as

$L_E = l[\frac{1}{4}[\frac{5}{2} + 16\phi^2][1 + 16\phi^2]^{\frac{1}{2}} + 3/32\phi \log_e [4\phi + [1 + 16\phi^2]^{\frac{1}{2}}]]$ .... (2a)
where ϕ is the dead load sag ratio .... f/l (see Figure 1).

$$W_I = \int_0^l \left[H_D \frac{ds}{dx} + \frac{1}{2} H_L \frac{ds}{dx}\right] d\lambda = \int_0^l \left[H_D + \frac{H_L}{2}\right]\frac{ds}{dx} \cdot H_L \frac{ds}{dx}\frac{ds}{AE_o} = \left(H_D + \frac{H_L}{2}\right)\int_0^l \frac{H_L}{AE_o}\left(\frac{ds}{dx}\right)^2 ds = \left(H_D + H_L/2\right)\frac{H_L}{AE_o}\int_0^l \left(\frac{ds}{dx}\right)^2 ds$$

The above equation contains the terms representing the horizontal component of cable stress for both dead and live loading; also the deflection Δ at any point in the span and the term "q" representing that portion of the total live load transmitted from the stiffening frames to the cables. With these factors evaluated, the cable stress for the loadings assumed can be readily determined. Moreover, since the *residual* live load active against the stiffening system is obviously (p-q) per unit length between points "a" and "b" and (–q) per unit length over the balance of the span, the stiffening frame stresses also will be determinate.

The above derivation constitutes the basic theory of stiffened suspension bridges and is comparatively simple. Its application, however, is not as simple as the theory itself inasmuch as the evaluation of the various unknowns is a process somewhat involved.

Equation 4 contains four unknowns; viz: H_D; H_L; q and Δ. and also the cable area "A" which obviously must be assumed for the solution, and subsequently corrected by a process of cut and try exactly as in the case of any other statically indeterminate frame. The term H_D is readily evaluated from statics as $\frac{wl^2}{8f}$ * where "f" is the dead load sag at center line span. This leaves for determination the terms q, Δ, and H_L each of which will be considered in order.

* The expression $H_D = \frac{wl^2}{8f}$ is mathematically correct only when the dead load per unit length is constant throughout the entire length of span. A constant deck load and a constant cross section of cable produces a combined unit dead load of varying intensity increasing from the center toward the ends due to the inclination of the cable. This discrepancy, for a 900-foot main span, was calculated by the authors and found to be approximately one-tenth of one per cent. The error introduced by using this formula is, in most instances less than the discrepancy existing between the actual weight of the structure and the most accurate estimate the designer can make; moreover, the variation in the unit dead load caused by the inclination of the cable is in many cases partially offset by a corresponding variation in the weight of the stiffening frame. The expression $H_D = \frac{wl^2}{8f}$ can therefore be used in designing any bridge of moderate length; however, excessively long-span structures should be investigated for this varying load.

$$-q = H_D\left[1+\frac{H_L}{H_D}\right]\frac{d^2\Delta}{dx^2} - \omega\left[1+\frac{H_L}{H_D}\right] + \omega$$

Article 2—Evaluation of the Term "q" (representing that portion of the live load transmitted to the cable)

Consider any infinitesimal increment of cable length as a free body in equilibrium (Figure 3) under the action of dead load forces only. $\Sigma v = o$ yields the following:

$$H_D\frac{dy_2}{dx} - H_D\frac{dy_1}{dx} = -wdx$$

But

$$dy_2 - dy_1 = d^2y$$

Whence

$$H_D\frac{d^2y}{dx^2} = -w \qquad (5)$$

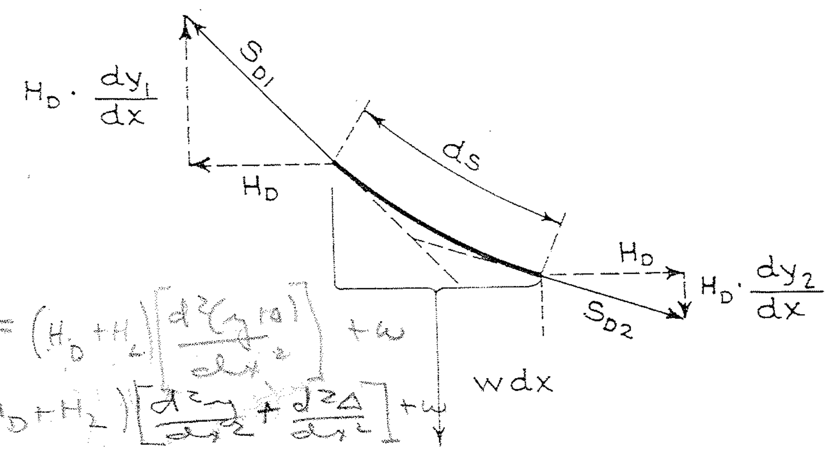
In a similar manner, after the application of live load, we may write for elastic equilibrium:

$V_2\ q$?

$$[H_D + H_L]\left[\frac{d^2(y+\Delta)}{dx^2}\right] = -(w+q) \qquad (6)$$

Whence

$$-q = H_D\left[1+\frac{H_L}{H_D}\right]\frac{d^2\Delta}{dx^2} - \left[\frac{H_L}{H_D}\right]w \qquad (7)$$



$$-q = (H_D+H_L)\left[\frac{d^2(y+\Delta)}{dx^2}\right] + \omega$$
$$= (H_D+H_L)\left[\frac{d^2y}{dx^2} + \frac{d^2\Delta}{dx^2}\right] + \omega$$

FIGURE 3

$$= (H_D+H_L)\left[-\frac{\omega}{H_D} + \frac{d^2\Delta}{dx^2}\right] + \omega = H_D\left[1+\frac{H_L}{H_D}\right]\left[-\frac{\omega}{H_D} + \frac{d^2\Delta}{dx^2}\right] + \omega$$
$$= H_D\left[1+\frac{H_L}{H_D}\right]\left[-\frac{\omega}{H_D}\right] + H_D\left[1+\frac{H_L}{H_D}\right]\frac{d^2\Delta}{dx^2} + \omega$$

I ALIGNMENT; DIRECTLY ABOVE EXISTING; LOWERED AFTER ERECTION AND DISMANTLEMENT OF EXISTING.

II LENGTH; 1920' (RE 2000' EXISTING)

III TYPE; DOUBLY SYMMETRIC SELF-ANCHORED SUSPENSION; 480', 960', 480' SPANS

IV FUNCTIONS: TWO 16' AUTO LANES TOP
ONE 10' RAIL LINE BOTTOM
TWO 8' BICYCLE/PEDESTRIAN LANES ON EACH SIDE

V TOWERS: 250' ABOVE PIERS

VI CABLES; SAG RATIO 1/6 TO DECK'S CENTROID

VII MATERIAL; A242 STEEL
30 KSI WORKING STRESS

VIII DESIGN METHODS: CLASSICAL ELASTIC
McCULLOUGH'S "BETA"

IX LOADINGS: DEAD 1610 lb/ft STRUCTURE
(GLOBAL) DECK 2770 lb/ft } QUASI-LIVE
RAILS 120 lb/ft }

X LOADINGS: TWO 50,000 LB AXLE SETS PER LANE
(LOCAL) ONE 60,000 LB RAIL CAR
100 LB/FT² BICYCLE-PEDESTRIANS

XI PROPERTIES OF TRUSS
VERTICAL MOMENT OF INERTIA: 78.0 FT⁴
LATERAL MOMENT OF INERTIA: 71.2 FT⁴

RADII OF GYRATION

9.13' VERTICAL; 9.19' LATERAL !

THEORETICAL EULER COLUMN LOAD

FOR SELF-ANCHORED 1920' LENGTH.

$k = 1$, $L = 960'$, $r = 9.16'$

$kL/r = 960'/9.16' = 105$

$F_E = k\pi^2 EI/l^2$

$= 3,235,000$ LB (1,618 TONS)

XII HORIZONTAL DEAD LOAD

$H_D = wl^2/8f$

w = dead load/ft = 4,500 lb/ft

l = span = 960 ft

f = sag = 960/6 = 160 ft

$H_D = 4{,}500 \times 960^2 / 8 \times 160$

$= 3,240,000$ LB

$H_D < F_E$ AS REQUIRED!

I STEEL DECK WEIGHS ≈ 25 lb/ft² + RAISES AERO FORCES ON SUSPENSION BRIDGES

II DEAD LOAD REDUCTION W/ STEEL DECK

A. KEEP STRINGER DESIGN FOR STIFFNESS

B. 25 lb/ft² × 32' = 800 lb/ft.

C. REDUCTION:

1. 4500 lb/ft − 2773 + 800 = 2527 lb/ft

2. 2527/4500 = 0.56 FACTOR

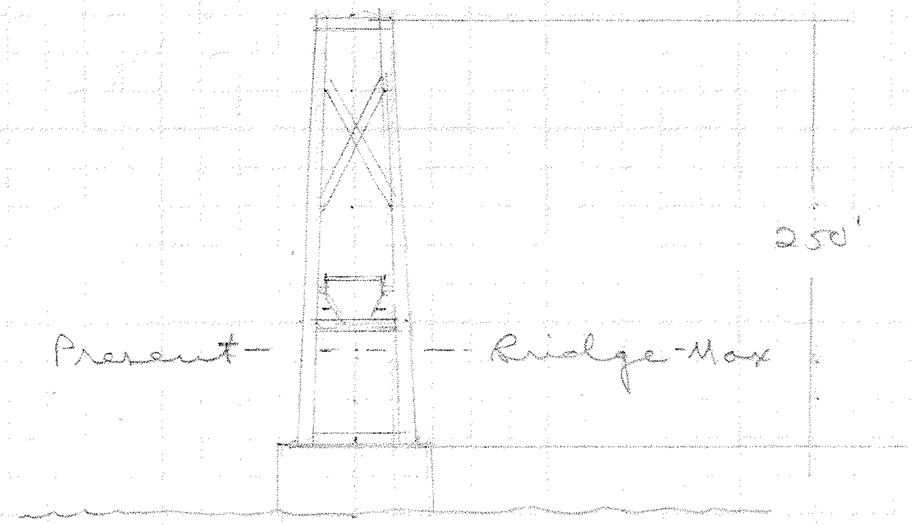
III HORIZONTAL DEAD LOAD IN CABLE

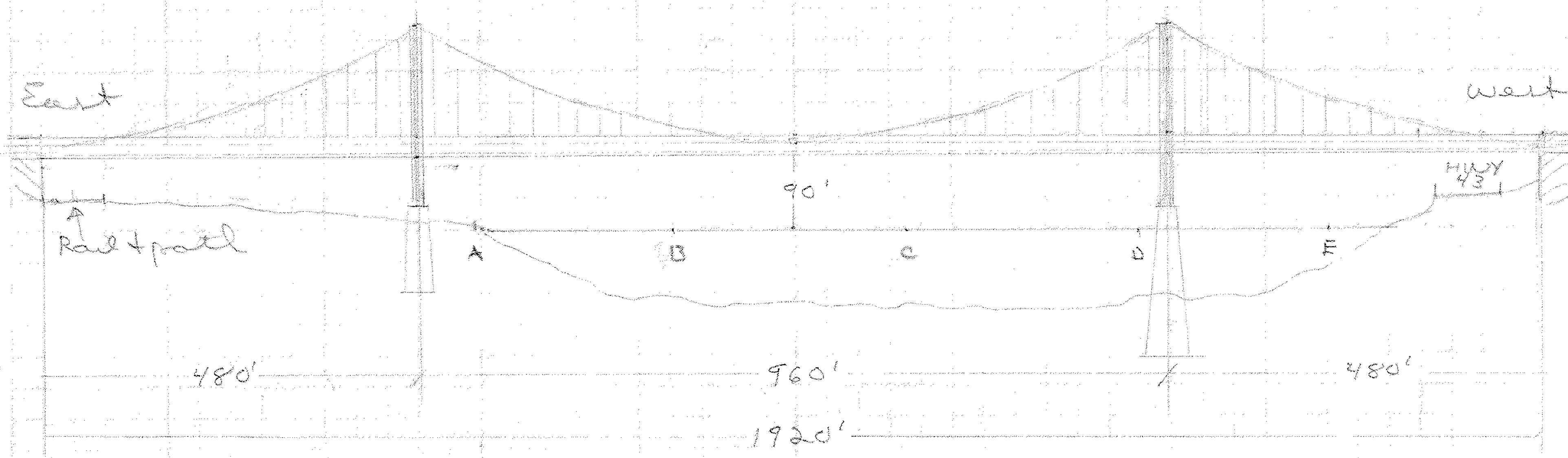
A. 3,240,000 lb × .56 = 1,817,440 lb

B. FRACTION OF ELMER LIMIT

1,819,440 / 3,325,000 ≈ 0.55 !

IV USE DIAGONAL STEEL DECKING





A, B, C, D, E are locations of existing piers.

I cf. CBM, #13, p 10. $H_D = wl^2/8f$

w = dead load = 4500 lb/ft

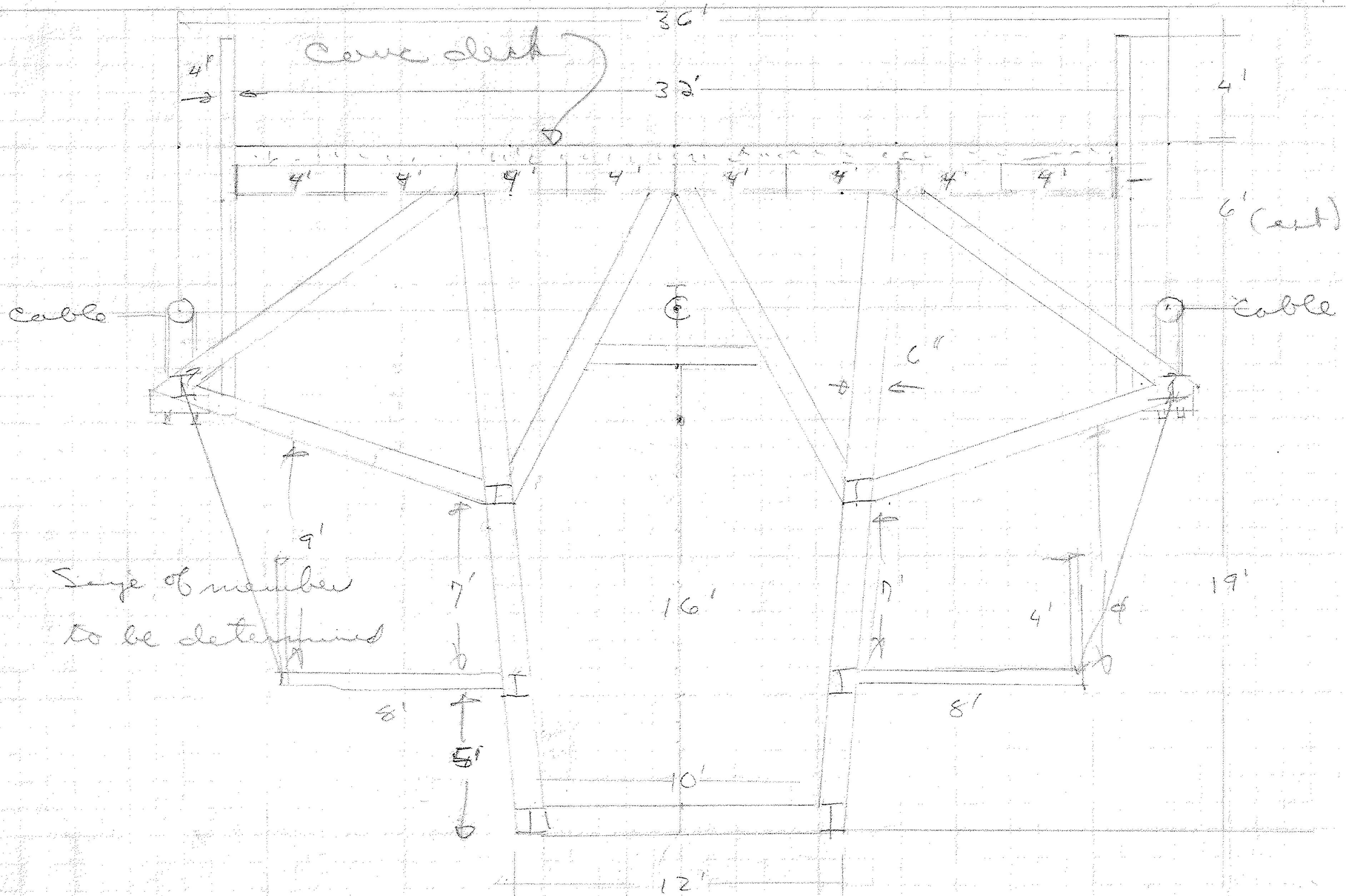
l = span = 960 ft

f = sag = 960'/6 = 160'

$H_D = 4{,}500 \times 960^2 / 8 \times 160$

$= 3{,}240{,}000$ lb (1,620 TONS)

H



STEEL BRIDGE

SECTION

1 Sept 2001

II MATERIAL: A242 "WEATHERING" STEEL

$\sigma_y = 50$ ksi; $\sigma_b = 30$ ksi; $\sigma_s = 20$ ksi

AISC 6th 5-91

($\sigma_b' = 0.60\sigma_y = 30$ ksi)

III ROAD DECK

A DECK: 8 in concrete on steel pan, 32 ft wide

1. DEAD: 130 lb/ft × (8/12) × 32' = 2773 lb/long ft

2. LIVE: 50 k/axle set in both lanes of 25 ft panel span, distributed among 3 stringers in each lane; 50 k × 2/panel = 100 k/panel

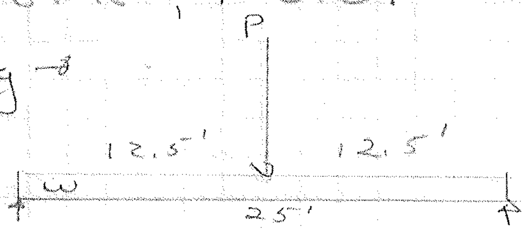
3. (DEAD + LIVE)/panel = 2773 × 25' = 69,333 lb
+ 100,000 lb
169,333/panel

B STRINGERS, 4' O.C.

← long →

AISC 6th 2-120, 122

1.



$w = (130\ \text{lb/ft})(8/12)\,4' = 347\ \text{lb/ft}$

$P = 50{,}000\ \text{lb}/3\ \text{stringers}$

2. $M_w = wl^2/8 = 347 \times 25^2/8$

$= 27{,}083\ \text{lb-ft}$

$M_p = Pl/4 = (50{,}000/3)(25/4)$

$= 50{,}000 \times 25/12$

$= 104{,}167\ \text{lb-ft}$

$M_w + M_p = 131{,}250\ \text{lb-ft}$

$M_T = 1{,}575{,}000\ \text{lb-in}$

FULL DEAD LOAD
4500 lb/ft (p7)
− 2770 lb/ft Deck
1730 lb/ft
− 120 lb/ft Rails
1610 lb/ft TRUSS dead load

EST TOTAL WEIGHT
1920' × 4500 lb/ft
= 8,640,000 lb

II B 3 $S_e > M_T / \sigma_f$

$= 1,575,000 / 30,000$

AISC 6th p2-4 $= 52.5\ in^3$; 12WF 45 ⇒ 58.2

There are effectively 8 of these

add $8 \times 45 = $ 360 lb/ long ft

Assume shear is ok

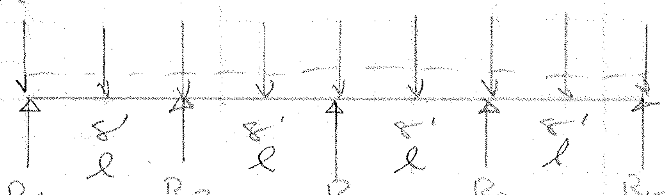
4. Net dead load/ long ft is now

2773
+360
3,133 lb/long ft

C Main cross girders, 32'

1. There are several different loads, but

+ width +



let us sum all live and dead loads and compute an equivalent uniform load for a 25' panel

i) Dead load of deck: (2773 lb/ft) x 25' = 69,325 lb

ii) Live load of 2 axle sets: 50,000 x 2 = 100,000 lb

iii) Total 169,325 lb

iv) Equiv. uniform load

AISC 6th 2-133 169,325 lb/32' = 5,291 lb/ft.

2. i) $M_{max} = |-0.1701 w l^2|$

$= 0.1701 \times (5291\ lb/ft) \times 8^2$

$= 57,604\ lb\text{-}ft$

II B 2 i) $M_{max} = 691,252$ lb-in

ii) $V_{max} = 0.607 w l$

$= 0.607 \times (5291$ lb/ft$) \times 8$

$= 25,693$ lb

3. $S_e \geq 6921,252$ lb-in$/(30,000$ lb/in$^2)$

$= 23.0$ in^3 ; 12B22 ⇒ 25.3 in^3

4. check shear

+ISC6th 1-22 i) Web is 12" × 1/4" = 3 in^2

2-119 ii) $25,693$ lb$/3$ in^2 $= 8,564$ lb/in^2

ok $< 20,000$ lb/in$^2 = \tau_a$

4 Net dead load per long ft is now 3,133 lb/ft

12B22 × 32' width/25' panel +28 lb/ft

3,161 lb/long ft

IV RAIL DECK

A DEAD

+ISC6th 1-107 1. RAILS: 2×60 lb/yd $= 2 \times 20$ lb/ft $= 40$ lb/ft

2. TIES: CONCRETE; 2' o.c.; ≈ 1 ft$^3 \times 150$ lb/ft^3

(150 lb/tie)/2' o.c. ≈ 75 lb/ft

3. RAILS + TIES 115 lb/long ft

B LIVE: 60,000 lb loaded car on 2 axle set

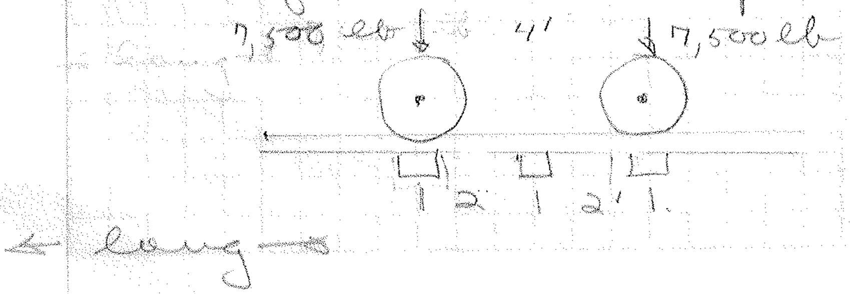
1. axles of each set ≈ 4' apart ; 60,000 lb/4 axles

$= 15,000$ lb/axle

(15,000 lb/axle)/

(2 wheels/axle)

$= 7,500$ lb/wheel



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PORTLAND CITY COUNCIL
COMMUNICATION REQUEST
Wednesday Council Meeting 9:30 AM

Council Meeting Date: November 10, 2010

Today's Date October 8, 2010

AUDITOR 10/08/10 AM10:22

Name JAMES B. LEE

Address 6016 SE MITCHELL, PORTLAND 97206

Telephone 503-771-6128 Email CADWAL@MACFORCEGO.COM

Reason for the request:

ECONOMIC DESIGN FOR
SELLWOOD BRIDGE

James B. Lee
(signed)

- Give your request to the Council Clerk's office by Thursday at 5:00 pm to sign up for the following Wednesday Meeting. Holiday deadline schedule is Wednesday at 5:00 pm. (See contact information below.)

- You will be placed on the Wednesday Agenda as a "Communication." Communications are the first item on the Agenda and are taken promptly at 9:30 a.m. A total of five Communications may be scheduled. Individuals must schedule their own Communication.

- You will have 3 minutes to speak and may also submit written testimony before or at the meeting.

Thank you for being an active participant in your City government.

Contact Information:

Karla Moore-Love, City Council Clerk
1221 SW 4th Ave, Room 140
Portland, OR 97204-1900
(503) 823-4086 Fax (503) 823-4571
email: **kmoore-love@ci.portland.or.us**

Sue Parsons, Council Clerk Assistant
1221 SW 4th Ave., Room 140
Portland, OR 97204-1900
(503) 823-4085 Fax (503) 823-4571
email: **sparsons@ci.portland.or.us**

1447

Request of James B. Lee to address Council regarding economic design for Sellwood Bridge (Communication)

NOV 10 2010

PLACED ON FILE

Filed NOV 05 2010

LaVonne Griffin-Valade
Auditor of the City of Portland

By ____________

COMMISSIONERS VOTED AS FOLLOWS:		
	YEAS	NAYS
1. Fritz		
2. Fish		
3. Saltzman		
4. Leonard		
Adams		