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The Motion of an Earthworm

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

ANTHROPOLOGY AND ALLIED SCIENCES

by

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THESIS

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Introduction

The purpose of this project is the design of an earth dam. The area caused to be flooded by the construction of this earth dam will be used in the improvement of the land given surrounding the flooded area. This land is to be platted and sold as resort lots.

The dam is located on Wolf Creek in section twelve, town five south, Cambridge township, Lenawee county, Michigan; approximately twenty miles south of Jackson Michigan on state road M-50.

It is readily accessible as it will be crossed by a private road which will serve as a means of transportation across the flooded area. This private road intersects the Springville road, a county road, one half mile north of the town of Springville located on State road M-50.

The water rights are controlled by the owners of the land to be inundated.

Design Data

In the investigation of this project it was necessary to establish a bench mark for convenience near the proposed site of the dam. No available United States Coast and Geodetic Survey bench mark was located in the vicinity. A substitute was obtained by using the road elevations on state road M-50 at the intersection of the Springville town road. A line of levels was run in to a point near the dam site and the elevation of the bench fixed. A line of zero azimuth was fixed between two points: the intersection of the present farm lane with the Springville town road to the east, and to the west a large oak approximately four feet in diameter.

Topography was shot in by the cross section method at twenty foot stations along the base line previously established and adjacent to the proposed dam site. A contour interval of one foot was chosen and the data tabulated in the field notes included with this thesis.

At the present time there are two small lakes connected by a creek of an average width of fifteen feet. The elevation of these lakes is 944.00 feet. The completion of the proposed dam will cause the elevation of the inundated area to be 940.00 feet, thus forming one lake of fifty acres in area.

Starting from a point located on the previously established base line a traverse was run in on the east side of the two present lakes by the stadia method. Choosing a contour interval of one foot due to the comparatively level land (composed mainly of marsh land) topography was taken to an elevation of 942.00. This data was tabulated in the enclosed field notes. Then proceeding from the original point a traverse was run in on the west side of the two present lakes by the same method and the data tabulated. Topography was also taken and the data tabulated and plotted by the use of the stadia plotting machine.

At the chosen elevation of 940.00 feet the topography of the land will give a rather sharply defined water line as the greater part of the area to be inundated is marsh land with sharply rising topography on either side. The cost of the dam being an important factor it was decided not to raise the level above the previously mentioned elevation as it would necessitate relocating the Springville town road making the total cost of the project prohibitive to the present owners of the property. Also since the elevation of the unusable land is below the proposed water level there seems to be no

necessity at present to increase the water elevation.

The soil was tested by means of a soil auger. Samples were taken at various intervals around the dam site and on a nearby hill to find suitable material for the earth fill necessary for the dam. The type of soil seemed to approximate a soft clay which will be suitable for this type of dam.

The velocity of the present stream, Wolf Creek, was measured by using small floats sent down dam. It was judged to be a representative cross section of the stream. A measured distance was laid off along the edge of the stream and the time interval of the floats recorded. Several readings were taken and recorded and the mean velocity calculated. The quantity was then calculated from the representative cross section of the stream and found to be three cubic feet per second. The calculations for the above will be found in this thesis under "Design Computations".

Difficulties encountered due to the fact that no contour maps of the area under consideration were available either from the state or county or the maps were restricted by obtaining from the United States Department of Agriculture aerial photographs. By the use of the stereoscope and the aerial photographs the maximum drainage area was obtained and sketched on their surface. The scale was computed and the drainage area measured with the planimeter. After the necessary computations were made the area to be drained was found to be 1.423 square miles. The characteristic of the drainage area is that it is long compared with the width, however, a large percentage is water surface being composed of a number of small connected lakes. The total area of the water surface is 327.5 acres as compared with an exposed land surface area of 719 acres. The large water area will tend to act as a storage basin and

offset the danger of possible floods due to a long narrow drainage area.

In the determination of the maximum rainfall for the area under consideration several factors were considered. Rain fall data was obtained from the West Lansing branch of the United States Department of Agriculture Weather Bureau. By investigating the maximum monthly rainfall of the various cities in the vicinity of the drainage area it was decided that the conditions recorded in the city of Adrian Michigan most closely approximated those of our drainage area. As no data was available for the district where our project was located by interviewing several farmers we were able to determine the weather conditions in the past, and from there we chose the city previously mentioned in order to have accurate records with which to work with in our computations. Rainfall data was obtained at Adrian Michigan which gave results covering a period from 1872 to 1930, giving a range of fifty two years. It was felt that a period of this length would include nearly all possibilities; and also the greatest maximum and lowest minimum values with a very excellent average precipitation values. However, as a check the city of Jackson Michigan, which is also quite near to our project, was investigated thoroughly for maximum and minimum values and annual average precipitation. The period was of shorter duration covering thirty three years from 1897 to 1930. The results given were comparable to those of Adrian with the exception of the maximum annual which was 31.01 inches in Adrian to 37.47 inches in Jackson. Therefore, in the design the results of Adrian will be used. Following are lists of the results of Jackson Michigan and Adrian Michigan giving the important monthly average, maximum and minimum recordings in various years.

PRECIPITATION

Adrian, Lenawee County, Michigan

Annual Average 34.71 Inches

Maximum Annual 64.01 Inches

Minimum Annual 21.59 Inches

Average Monthly		Maximum Monthly	Minimum Monthly
Jan. 1.93 inches	(1890)	5.15 inches	(1917 - 1904) 0.40
Feb. 2.14	(1891)	6.51	(1907) 0.20
Mar. 2.50	(1892)	5.40	(1910) 0.24
Apr. 2.82	(1890)	5.25	(1899) 0.52
May. 3.93	(1892)	6.13	(1903) 1.37
June. 3.75	(1903)	10.03	(1904) 0.70
July. 3.15	(1897)	7.50	(1913) 0.24
Aug. 3.08	(1926)	6.52	(1894) 0.48
Sept. 3.37	(1926)	7.17	(1908) 0.32
Oct. 2.65	(1881)	10.71	(1892) 0.46
Nov. 2.92	(1881)	8.93	(1917) 0.21
Dec. 2.40	(1881)	3.57	(1919) 0.11

PRECIPITATION

Jackson, Jackson County, Michigan

Annual Average 31.89 Inches

Maximum Annual 37.47 Inches

Minimum Annual 20.70 Inches

Average Monthly	Maximum Monthly	Minimum Monthly
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Jan. 1.94 inches	(1929) 3.69 inches	(1919) 0.32
Feb. 1.30	(1908) 5.25	(1914) 0.57
Mar. 2.30	(1913) 3.77	(1910) 0.13
Apr. 2.67	(1919) 5.92	(1899) 0.09
May 3.33	(1905) 6.12	(1925) 0.71
June 7.40	(1907) 7.60	(1913) 0.34
July 6.39	(1923) 7.11	(1930) 0.22
Aug. 2.30	(1926) 7.82	(1903) 0.67
Sept. 7.16	(1926) 7.43	(1909) 0.73
Oct. 2.63	(1898) 5.23	(1924) 0.31
Nov. 2.72	(1927) 5.03	(1904) 0.07
Dec. 1.84	(1920) 3.72	(1900) 0.28

It is evident that not much emphasis need be placed on monthly minimums in relation to the effect of possible minimum flow in the stream. It is considered the annual maximum and minimums will give a much truer value for volume of water to be expected.

The computation for the maximum flow was accomplished in the following manner. The maximum rainfall for a seventy two hour period was 6.33 inches. It is considered that a seventy two hour period will approximate an unusual condition of flow. For a period of twenty four hours a value of 0.7 of the seventy two hour period will be used. The value obtained by computation gave a total flow of one hundred and seventy five cubic feet per second. However, due to the loose texture of the permeable soil the percolation in the drainage area will be high and a factor of 0.2 was thought to be quite on the safe side. Also due to the large water surface of the area a large rainfall would cause only a correspondingly small increase in the level of the lakes. Therefore, with these factors considered it was decided that a maximum flow of one hundred cubic feet per second would be the limiting factor in our design of the spillway capacity.

Evaporation from water surface being 3.5 inches per month and evaporation and transpiration on land being 2.33 inches per month, there will be an increase of 0.37 inches evaporating from the newly inundated land. Using the additional water surface and this 0.37 inches, the increased water lost will be 91,750 cubic feet per month. With the average flow of three cubic feet per second in the stream at present, the safety factor against evaporation exceeding flow is 85.

The design of the dam was governed by several factors. The slope of the fill which was chosen as three to one on the upstream face and a two and one half to one on the downstream face. These slopes are very much on the safe side and have been proven on other dams of a type similar to this one which were investigated by the authors. The dam also must have a roadway across the crest as it will be used as the means for cottage owners to reach their property in the prospective subdivision. We chose a road twelve feet wide with three feet allowed on each side for shoulders and as protection against the possibility of crushing the upstream face of the dam should a vehicle be forced off the road..... It was assumed that the traffic would be light as the road would be private and hence available only to property owners which will not be too great in number. Therefore, we felt that our dimensions were more than ample in the design of the crest of the dam.

It was necessary to construct a reinforced concrete bridge to span the spillway of the dam. The design of this bridge will be taken up in a latter part of the thesis.

The flow width was computed and found to be 8.66 feet. Therefore, we may again assume that a crest of eighteen feet our dam crest is quite safe against any possible factors we might consider in the design. The computations for the above will be found under "Design Computations".

The possibility of failure due to over topping by excessive wave action was checked on our proposed lake and found to give a value for freeboard of 1.5 feet using a reservoir of approximately one quarter mile in length. This factor will be incorporated in the total height of the earth dam.

The earth structure was tentatively sketched on our large scale

contour top of the dam site at an elevation of 848.00 feet. This elevation will give us the desired height of water surface, 846.00 feet, and also apply free board for protection against overtopping. The line of intersection of the upstream and downstream slopes was plotted and from this we computed the volume of fill necessary. The volume of earth was computed and found to be 997 cubic yards. The location of the dam with regards to the proposed roadway was investigated with the help of the owners of the property and our first choice was considered favorable for the situation presented. Therefore, the design will be continued along this line using a crest elevation of 842.00 feet.

The material to be used as fill in the dam will be procured from a nearby hill which now is a traffic hazard on the Springville town road. The fill material is very fine, therefore, having a comparatively steep slope for the saturation line. Using a slope line of twenty feet in one hundred the line of saturation will intersect the bottom sixteen feet from the downstream toe of the dam. From this it can be seen that the dam is safe from water coming through the downstream face of the dam. However, a concrete core wall as a protection against the possibility of rodents burrowing and causing failure of the dam, will be constructed down to medium clay below the proposed lake bed. In addition to the protection the core wall affords against rodents additional protection will be given by the pipes against saturation of the soil although no provision is made for this in the core wall design. Computations for line of saturation will be found under "Design Computations".

In testing the soil for bearing a value of 120 pounds per cubic foot was judged to be safe and taking the height of the earth dam to be eight feet a total of 960 pounds per square foot was the maximum

bearing pressure. However, for 40,000 pounds per square foot is a safe value for a soft clay of this type. Then, again, with this in mind a safety factor of 2 is obtainable and proves ample in this case. The core wall previously described must go down to wetting clay with a bearing capacity of 2000 pounds per square foot which is quite enough for our core wall design.

Sliding was investigated and a safety factor of ten will be used as our limiting factor. The total weight per lineal foot of fill is 40,000 pounds as against 7,830 pounds per lineal foot of water pressure giving a total downward pressure of 47,830 pounds per lineal foot. We will consider a coefficient of one for sliding friction. Therefore, a force of 47,830 pounds will be necessary before sliding takes place. The horizontal force of the water will tend to push against the retarding force of 47,830 pounds per foot of fill is 187.5 pounds per foot giving a safety factor against sliding of 257 which is definitely on the side of safety. Computations for the slope will be found under "Design Computations".

The spillway design considered the following factors. Simplicity of design and the maximum quantity of flow to be handled. It was decided by the authors that the open channel would give the best results as the total cost of the project was an imperative limiting factor. Therefore, the open channel construction will give inexpensive form work and hence save on an important cost item. The maximum quantity of flow to be handled was previously computed and found to be 120 cubic feet per second. A total height of the open channel of one foot was a limiting factor and from this a velocity head of one foot was assumed. The velocity computed from this head was 3.61 feet per second. Then from the formula $Q = A \times V$, an area of approximately thirteen square feet was found to

be sufficient on the left side. This gave a length of spillway of fifteen feet on the upstream side. In order to flatten out down casts the amount of energy was reduced by narrowing down the open channel at the downstream end a distance of of twenty four feet horizontally from the upstream face. A velocity of fifteen feet per second was assumed giving an area of 8.47 square feet. Using the Manning formula for open channels with a roughness coefficient of 0.014 the slope per foot of the water surface was computed. A spillway length of six feet at the downstream end will be used. Continuing down from a point directly below the intersection of the crest of the dam with the downstream face the spillway has a slope of 0.41 feet per foot horizontally with a width of six feet. This increased slope will give a total velocity of fifty seven feet per second at the base of the spillway. Therefore, precautions will be taken against possible damage of the spillway at the toe. A concrete apron will be extended as shown in the accompanying drawings. This apron will keep the water from seeping under the toe causing the soil to wash out of the earth dam. In order to dissipate the energy developed by the high water velocity above rubble will be placed in the concrete on the surface of the downstream spillway. The dimensions for the concrete on the spillway, and computations for the spillway will be found under "Design Computations".

The outlet for the structure will be an eight inch galvanized cast iron pipe extending through the dam at the base, and circumscribed by two concrete collars, one the core wall, a distance from the centerline of the dam. The concrete collars are a protection against water flowing along the surface of the pipe. The pipe will keep on either side of the stream and will continue vertically upward to the

elevation of the proposed water surface. This outlet will also furnish additional spillway capacity in case of any emergency. Due to the small normal flow, three cubic feet per second, the eight inch pipe proved ample as an outlet device. As was previously mentioned cost is a consideration and the pipe method of outlet was chosen as the inexpensive way of solving the problem.

The design of a reinforced concrete bridge to cross the spillway was carried out according to accepted practice and the computations and drawings will be found under "Design Computations".

COST ESTIMATE

The estimated cost of the project is as follows:

Cost of fill plus allowance for consolidation:

$$\begin{aligned} 987 \text{ cubic yards} + 5\% \times 987 \text{ cubic yards} &= 1037 \text{ cubic yards} \\ 1037 \text{ cubic yards} \times \$0.50 &= \$518.43 \end{aligned}$$

Cost of form work:

Allowing \$0.15 per square foot

Total area = 4,062 square feet

$$4,062 \times 0.15 = \$609.30$$

Less 83.00 for reusing lumber

$$\text{Total} \quad \$526.30 \quad 526.30$$

Cost of steel:

\$0.07 per pound; placing steel \$25.00 per ton

$$2,807 \text{ pounds} \times 0.07 + \frac{2,807}{2,000} \times \$25.00 = 231.00$$

Cost of concrete:

Concrete cost \$12.00 per cubic yard

$$\frac{1,485.6 \text{ cu. ft.}}{27 \text{ cu. ft.}} \times \$12.00 = 660.00$$

Cost of outlet:

Total cost of cast iron pipe galvanized 115.00

plus 3 inch elbow

Total estimated cost of the project **\$2037.43**

DESIGN COMPUTATIONS

Area of land to be inundated in acres:

$$\frac{2,153,000}{43,560} = 49+ \text{ say } 50 \text{ acres}$$

Drainage area from aerial map:

$$\frac{41.3 \times 10^6 \text{ sq. ft.}}{(1,100)^2} = 1.433 \text{ sq. mi.}$$

Area covered by water at present:

$$4.57 \times \frac{(20,000)^2}{(11)^2} = 9.31 \times 10^6 \text{ sq. ft.}$$

$$\frac{9.31 \times 10^6}{43,560} = 213.5 \text{ Acres}$$

Area of land exposed at present:

$$(41.30 - 9.31) \times 10^6 = 31.99 \times 10^6 \text{ sq. ft.}$$

$$\frac{31.99 \times 10^6}{43,560} = 739 \text{ Acres}$$

Precipitation:

The amount of water falling per year:

$$\frac{41.30 \times 10^6 \times 40.70}{12} = 13.8 \times 10^8 \text{ cubic ft. per year}$$

Computation of maximum flow:

24 hr. period factor of 0.7 x 70 hr. period

$$0.7 \times 0.32 = 4.336 \text{ inches in 24 hrs.}$$

$$\frac{(4.336) \times 1.433 \times 57.2 \times 10^6}{12} = 15.13 \times 10^6 \text{ cubic feet per 24 hrs.}$$

$$\frac{15.13 \times 10^6}{24 \times 60} = 1.75 \times 10^5 \text{ cubic feet per sec.}$$

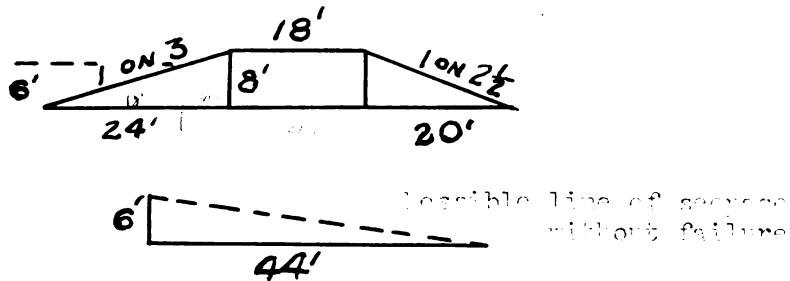
A factor of 0.2 for ground absorption

$$175 \text{ cu. ft. per sec.} \times 0.2 = 35 \text{ cu. ft. per sec.}$$

$$175 \text{ cu. ft. per sec.} - 35 \text{ cu. ft. per sec.} = 140 \text{ cu. ft. per sec.}$$

$$= 100 \text{ cubic feet per second maximum flow}$$

Line of saturation:



$$\frac{6}{44} = \frac{X}{100} ; X = 13.6 \text{ feet: } 13.6 \text{ feet drop in } 100 \text{ feet slope of possible line of seepage}$$

Sliding:

Coefficient 1.0

$$\text{Wt. water } \frac{6 \times 18}{2} \times 62.5 = 3,380 \text{ pounds per lineal foot}$$

$$\text{Wt. earth } \frac{(6 \times 20)}{2} + (8 \times 18) + \frac{(8 \times 24)}{2} \times 125 = 40,000 \text{ Pounds per lineal foot}$$

$$\text{Total downward force} = 40,000 + 3,380 = 43,380 \text{ pounds per foot Before sliding takes place.}$$

$$\text{Horizontal force of water: } \frac{6 \times 62.5}{2} = 187.5 \text{ Pounds per lineal foot}$$

$$\text{Sliding factor of safety: } \frac{43,380}{187.5} = 231$$

Foreboard needed for wave action:

$$h = 1.5 (D)^{\frac{1}{3}} + 2.5 - (D)^{\frac{1}{4}}$$

D * Length of reservoir
in miles

$$D = \frac{1}{4} \text{ mile}$$

$$h = 1.5 \left(\frac{1}{4}\right)^{\frac{1}{3}} + 2.5 - \left(\frac{1}{4}\right)^{\frac{1}{4}}$$

$$h = 1.5 \text{ feet}$$

Top width:

$$W = 2 (H)^{\frac{1}{2}} + 3$$

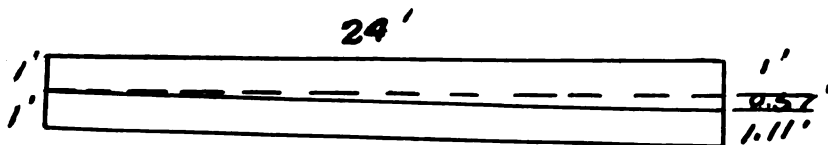
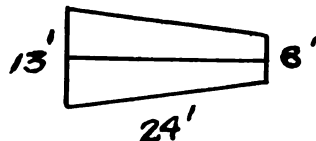
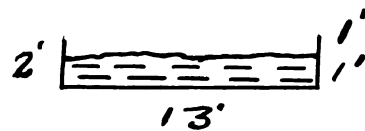
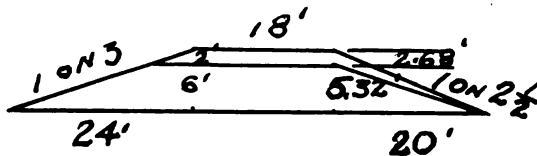
W * Top width in feet

H * Maximum height of embankment in feet

$$W = 2 (8)^{\frac{1}{2}} + 3$$

$$W = 8.16 \text{ feet}$$

Spillway design:



Design of spillway continued:

Maximum Q = 100 c.f.s. Design as open channel

Allowing a velocity head of 1.0 ft.

$$V^2 = 2 g h = 8.04 \text{ ft. per sec.} \quad \text{Say } 8.0 \text{ ft. per sec.}$$

$$A = \frac{Q}{V} = \frac{100}{8} = 12.5 \text{ sq. ft.} \quad \text{Say } 13.0 \text{ sq. ft.}$$

Will give a spillway length of 13.0 feet upstream side

Assume velocity of 15 ft. per sec. at downstream end

$$A = \frac{100}{15} = 6.67 \text{ sq. ft.}$$

Use spillway length of 6.0 feet at downstream side

Using Manning formula for open channels.

$$V = \frac{1.486 r^{2/3} s^{1/2}}{n}$$

$$n = 0.014$$

$$r = 0.812$$

$$V = 15 \text{ ft. per sec.}$$

Solving for "s"

$$s = \left[\frac{15 \times 0.014}{1.486 \times (0.812)^{2/3}} \right]^2 = 0.0038$$

$$21.0 \text{ ft.} \times 0.0038 = 0.57 \text{ ft. drop in water surface in 21 ft.}$$

Downstream spillway:

$$\text{Average "r"} = 0.801$$

$$V = \frac{1.486 \times (0.82)^{2/3} \times (0.041)^{1/2}}{0.014} = 42 \text{ ft. per sec.}$$

Total velocity at base of downstream open channel

$$42 + 15 = 57 \text{ ft. per sec.}$$

LOADING DIAGRAMS

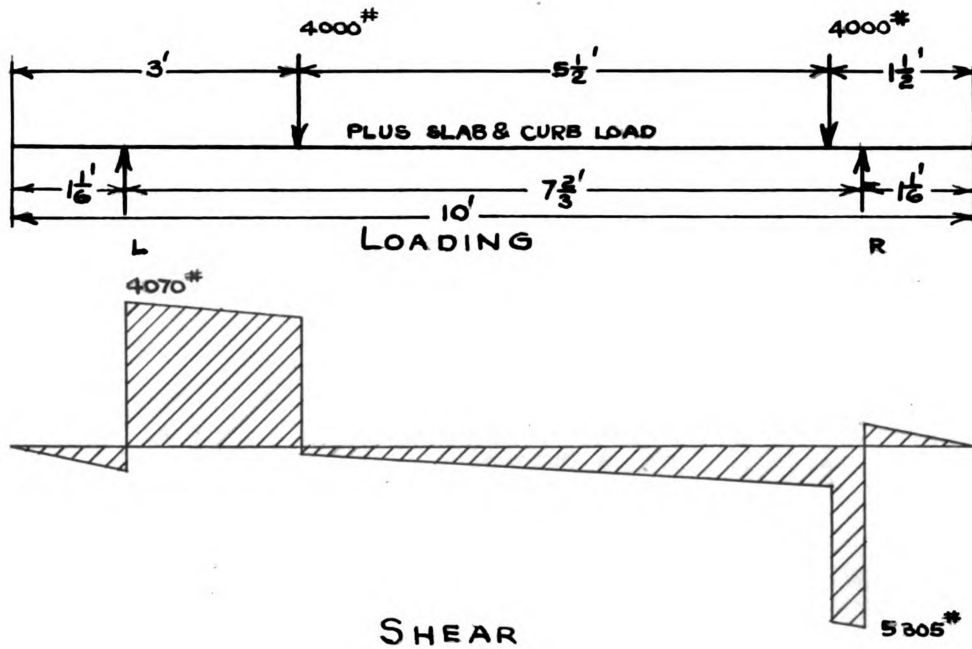


FIG. 1

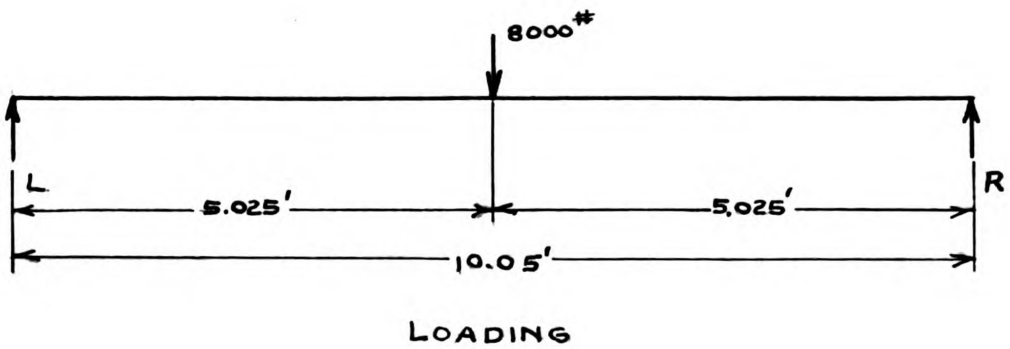


FIG. 2

Design of reinforced concrete bridge problem 2:

Referring to Fig. 1

$$4000(15/6 + 7^{1/3}) - 7^{2/3} R = 0$$

$$R = 1720 \text{ pounds} = V \text{ (live load) (shear at left end of right span per foot)}$$

Wt. per foot dead load

$$150 = (1 \times 2/3 \times 2 + 10 \times 1 \times 1 + 2 \times 1 \times 2/3) = 1700 \text{ pounds per foot}$$

Compute dead shear at left end of right span.

$$\frac{1 \times 7 \times 150}{6} = 525 \text{ pounds} = V \text{ (dead shear per foot)}$$

Referring to Fig. 2

Sum of M to left of "0"

$$= (2 \frac{3}{4})(150 \times \frac{2}{3} \times \frac{1}{2}) + (1 \frac{5}{16})(150 \times \frac{1}{2} \times \frac{2}{3}) + (150 \times 3 \times 1) \times \frac{3}{2} - (1 \frac{5}{16})(4070) = 2,556.16 \text{ ft. pounds}$$

$$d(\text{required}) = \sqrt{\frac{M}{R_b}} = \sqrt{\frac{6,556.16 \times 12}{151 \times 12}} = 6.4 \text{ - Say 7 inches}$$

Total depth of beam is 12 inches therefore safe

Investigate R.M. when loaded symmetrically

$$L = R = \frac{1,000 + 1,700}{2} = 4,850 \text{ pounds}$$

Max. R.M. occurs in middle of beam

$$\begin{aligned} \text{R.M.} &= (4 \frac{3}{4})(150 \times \frac{2}{3} \times \frac{1}{2}) + (3.833)(150 \times \frac{1}{2} \times \frac{2}{3}) + (2.5) \\ &\quad \times (150 \times 5 \times 1) + (2)(1,000) - (3.833)(4,070) \\ &= 5,276 \text{ ft. pounds (smaller than previous R.M.)} \end{aligned}$$

Investigate steel necessary:

$$A_s = 0.0031 \times 12 \times 7 = 0.7896 \text{ sq. in. per foot}$$

$$7/8 \text{ inch round bars at 9 inches} = 0.80 \text{ sq. in.}$$

This steel runs perpendicular to centerline of bridge.

(See Fig. 2)

Total length of bars:

$$81 + 2 \times 12 + 2 \times 5 = 105 \text{ inches; } 11 \text{ feet} - \text{one inch}$$

Reinforcement steel to run lengthwise of bridge:

$$0.002 \times 12 \times 10 = 0.24 \text{ sq. in.}$$

$$\text{Use } \frac{1}{4} \text{ inch square bars at 12 inches} = 0.35 \text{ sq. in.}$$

Check shear at left edge of right stem:

$$V \text{ (total)} = 4720 + 525 = 5,245 \text{ pounds}$$

$$v = \frac{V}{A} = \frac{5,245}{12 \times 0.77 \times 12} = 42.3 \text{ pounds per square inch}$$

$$(\text{use } "j" = 0.37) \quad 50.0 \text{ pounds allowable}$$

R due to dead load:

$$\frac{1,700}{4} = 425 \text{ pounds}$$

$$\text{Total R} = 425 + 4,720 = 5,145 \text{ pounds}$$

Referring to Fig. 2:

$$5.025 \times 4,000 = R \times 10.05 = 4,000 \text{ pounds}$$

$$\text{Live load B.M. } 5.025 \times 4,000 = 20,100 \text{ ft. pounds}$$

$$\text{Dead load B.M. } 1,700 \times 10.05 \times \frac{10.05}{4} = 43,000 \text{ ft. pounds}$$

$$\text{Total B.M. } 43,000 + 20,100 = 63,100 \text{ ft. pounds}$$

$$\text{B.M. per beam} = \frac{63,100}{2} = 31,550 \text{ ft. pounds (use } "j" = 0.31)$$

$$"d" = 18 - 2 = 16 \text{ inches}$$

$$31,550 \times 12 = 18,000 \times A_s \times 0.91 \times 16$$

$$A_s = 1.442 \text{ sq. in.}$$

(See Fig. 2)

Referring to Fig. 2:

$$\frac{x}{2} (28 x) - (1.57 \times 15)(16 - x) = 0$$

$$x = \frac{-b \pm \sqrt{b^2 - 4 a c}}{2 a} \quad x^2 + 1.672 x - 23.85 = 0$$

$$x = 4.4 \text{ inches}$$

$$T = C = \frac{P}{2} \times 1.44$$

$$T = 18,000 \times 1.44 = 25,920 \text{ pounds}$$

$$25,920 = f_c \times \frac{4.4}{2} \times 28$$

$$f_c = 426 \text{ pounds per square inch; (allowed 300 pounds per sq. in. therefore O.K.)}$$

Referring to Fig. 2:

Investigate bearing of stems on walls of spillway:

Use spillway walls of 8 inches; stems are 8 inches

Bearing for one stem under maximum load:

$$\frac{(1,700 \times 10.05)}{4} + 1,730 = 9,050 \text{ pounds}$$

$$\text{Area available: } 8 \times 8 = 64 \text{ sq. in.}$$

$$\text{Bearing stress} = \frac{9,050}{64} = 142 \text{ pounds per sq.in. (325 pounds per sq. in. allowed)}$$

Investigate shear at section "d - d":

$$\text{Area} = b \times d = 56 \times 6$$

$$\frac{9,050}{56 \times 6} = 26.9 \text{ pounds per sq. in. (50 pounds per sq. in. allowed)}$$

Top Slab and Downstream Runway:

Temperature steel:

$$6 \times 12 \times 0.002 = 0.144 \text{ sq. in. Needed}$$

$$\text{Use steel both ways } \frac{7}{8} \text{ in. round bars at 9 in.} = 0.15 \text{ sq. in.}$$

Temperature steel in downstream runway (sides) same as for bottom because same crosssection.

$$8 \times 12 \times 0.002 = 0.192 \text{ sq. in. Needed}$$

$$\frac{1}{2} \text{ in. round bars at 11 in.} = 0.21 \text{ sq. in.}$$

(Same spacing for steel needed for floor of spillway on top.)

Core Wall:

Temperature steel:

$$A_s = \text{psd}$$

$$A_s = 0.002 \times 12 \times 8 = 0.150 \text{ sq. in. per foot}$$

Use $\frac{1}{4}$ in. round bars at 10 in. spacing both horizontal and vertical.

Packet has: 1 - Photos

122

227

TMS

Photo 1

SUPPLEMENTARY
MATERIAL

122
227
TMS
Map



6-22-37

XV-4-327

A
R
E
A

D
R
A
I
N
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G
E



6-22-37

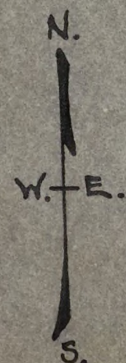
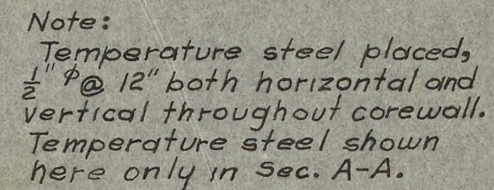
XV-3-275

DRAINAGE

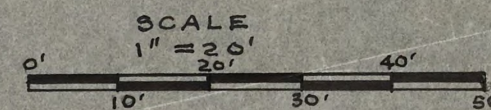
AREA



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SLOPES
UPSTREAM 1 ON 3 ; DOWNSTREAM 1 ON 2.5



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