STRUCTURAL PROBLEMS IN THE DESIGN OF A LOW-HEAD POWER PLANT

THESIS FOR THE DEGREE OF M. S.

Howard James Berkel

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STRUCTURAL PROBLEMS IN THE DESIGN OF A LOW-HEAD POWER PLANT

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THESIS

PREFACE

Through the courtesy of the Allied Engineers, Incorporated, an organization associated with the Consumers Power Company, Jackson, Michigan, the basis for the actual design contained in this thesis was furnished. This consisted of a map of a proposed location for a low-head power development, and certain data relative to stream flow and foundation conditions. This material was furnished by Mr. Edward M. Burd, Civil and Hudraulic Engineer.

The writer is also greatly indebted to Professor C. L. Allen, head of the department of Civil Engineering, Michigan State College, for his supervision of the preparation of this thesis; also to Mr. Wylie Bowmaster and Professor W. W. Hitchcock for helpful advice and contributions to the material.

Howard James Berkel

East Lansing, Michigan

June 1933

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CHAPTER 1

THE DEVELOPMENT OF WATER POWER IN THE UNITED STATES

Records dating back to the time of the ancients give the first evidence of the use of water power by crude devices for purposes of irrigation, and for the performance of other various rather simple applications. Some forms of float wheels, constructed of bamboo, are still in use in China, while other very crude wheels, constructed of timber, may still be seen in other foreign countries, having been preserved for their historical interest.

The coming of the breast, overshot, and undershot wheels marked a distinct advance over the more primitive types. These were, in effect, gravity wheels and efficiencies ranging from 30% for the undershot, to 80% for the overshot, were not uncommon.

The development of the hydraulic turbine in the middle of the nineteenth century revolutionized the use of water power and resulted in the superseding of the overshot wheel. The overshot wheel, even though it had a relatively high efficiency, was limited in its use to heads somewhat below 50 feet.

The periods between the middle and the end of the nineteenth century saw the development of the American, or mixed, or inward-flow turbine in America, and small developments were superseded by those utilizing single heads over 20 feet. Soon after 1890, the use of the

American wheel developed to such an extent that our own wheel manufacturers were building wheels of the reaction type to be used for heads of 500 feet and more.

The impulse or Pelton wheel was developed and used efficiently for high heads, especially in the West.

From 1900-1910 greater speed and power were obtained both here and abroad by the use of several wheels on a single shaft. Better wheel design and the use of vertical generator units, and more suitable thrust bearings for large units, have brought about the use of the single runner vertical units at the present time.

The period of the World War, with its great boom in manufacture, caused a very noticeable increase in the demand for water power in this country. A contributing factor was the increase in the cost of fuel and uncertainty in delivery due to labor unrest. This popularity still exists today in this country, water power possessing far greater appeal than any other form of energy generation. Hydraulic turbines are being improved constantly, and in order that efficiencies well over 90% may be obtained, great care has been given the design of such features as wheel settings, flumes, and draft tubes.

Accompanying the development of water power equipment has been an increase in the capacity and radius of practicable transmission of power. The contrast between 40,000 volts in the year 1900, and 220,000 volts only twenty-four years later is evidence of this rapid advance. A comparison of the lengths of lines used in the earlier days extending distances of a few miles, and at the present time distances up to 300 miles, will serve also to illustrate the extent of this progress.

With regard to the present status of water power in the world, a conservative authority estimates that the potential water power available is four times the total amount of the present use of this resource. A total potential water power of 439 million horse power in the world, the present use of this resource is about 2 or 3 percent of the total potential power.

Quoting from the estimates of the United States Geological Survey which states that for the United States alone, area 3,026,791 square miles, the developed horse power is 2.97 horse power per square mile, and the potential is 11.6 horse power per squaremile.

From a statement by the same authority*, Europe as a continent leads with 2.29 developed horse power per square mile. Yet the United States, with about .8 of the area of Europe, materially exceeds the latter with about .68 horse power per square mile of area.

Most of the older plants in the United States , excluding those in the middle West have been developed under laws similar to the Mill Act in New England and New York. This act stated that a power site owned by an individual or a corporation may be developed and, if necessary, flowage made of the land or undeveloped water rights upstream. Should riparian owners refuse to sell their rights to the land or flowage, these could betaken by right of eminent domain.

In the West, a great portion of the available water

* Barrows, Water Power Engineering.

power is located on the public forest lands. For many years it was necessary to receive congressional sanction, a slow tedious process, to obtain permits for the development of such territory since no fixed policy prevailed. President Roosevelt in two messages to Congress, one in 1908 and another in 1909, vetoed acts conferring franchises for the development of water power in the public lands. The president stated, " that adequate provisions for the safeguarding of the general public had not been incorporated in the acts, that no rights involving water power should be granted to any corporation in perpetuity, but only for such length of time as to allow them to conduct their business profitably, and privileges obtained from the National Government should be paid for by a reasonable charge." Action was taken by Congress in the passage of the amended Water Power Act, on March 3, 1921, which excluded all national parks from its provisions.

The Federal Water Power Commission, whose purpose is, " to provide for the improvement of navigation, the development of water power, and the use of the lands of the United States in relation thereto," consists of the Secretaries of Agriculture, Interior, and War. The commission is given the power to investigate the cost of water power development, its availability for market, and its fair value in any region to be developed, and in these duties the commission is of prime importance in the development of water power in this country.

Licenses are issued to citizens, municipalities, or states to construct and maintain projects for the development of navigation and for the development, transmission, and utilization of power, " (1) from or in any of the navigable waters of the United States, and (2) upon any part of the public lands or reservations, and (3) to utilize the surplus water from any government dam."* These licenses are issued for a period not to exceed 50 years and preference is given to states and municipalities as far as possible. The annual fee is fixed and collected by the Federal Water Power Commission and is used to cover the cost of administration of the act and as payment for the use of national lands or other property. Such other rules and regulations not of especial significance in this thesis, may be found in complete text available for distribution in pamphlet form.

Data available up to June 1925 list 524 applications to the Federal Water Power Commission for power developments and more than 100 applications for transmission lines. The applications involve more than 240 million horse power. During the year 1925 alone, for example, an aggregate of 620,000 horse power in 80 applications for power projects, and 32 for transmission lines, were filed with the commission which illustrates the extent of its activities.

The super-power investigation was made during 1920-1921 under the direction of the Geological Survey, and is reported in Paper 123. This investigation showed that, considering the super-power zone as including the New England states, New York, Delaware, Pennsylvania, Maryland, there was a concentration of nearly 25% of the population of the United States, having 315 electric utilities, 18 railroads, and 96000 industrial plants. Only one-fifth of the required power of this zone can be supplied by water power. Quoting

* Barrows, "Water Power Engineering".

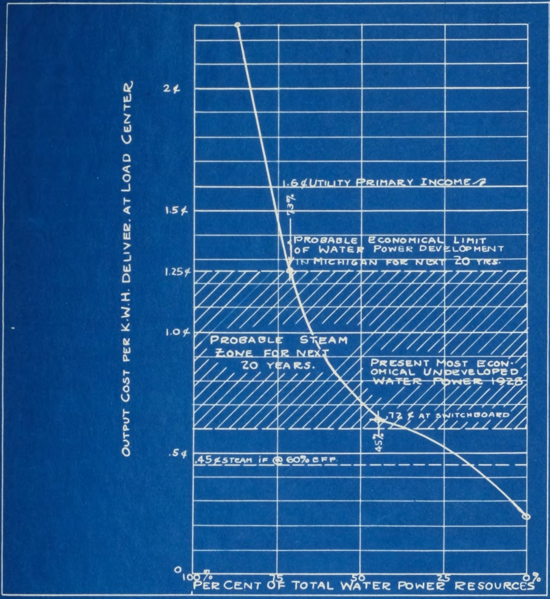
from Paper 123, the survey recommended, " that large steam plants be located at tidewater and on inland waters, as well as being bolstered by utilization of hydro-electric power which may be obtained from rivers in or adjacent to the territory." A system of interconnected transmission lines of high voltage, involving a combined capital investment of $ 1,300,000,000, which would net nearly 33% on the investment above the fixed charges is estimated as being necessary to take care of the Super-power. The activities of the United States Geological Survey in making this investigation have done much toward the extensive use of the feature of super-power development.

CHAPTER 11.

THE PRESENT STATUS OF WATER POWER IN MICHIGAN

The first water power developments in the Northwest territory were small local mills patterned after those built by the pioneers coming largely from New England and New York state. Lack of transportation was the controlling factor in all these projects. The dam, wheel, mill, and even some of the machinery were built of local resources, and the output of the mill, whether lumber or grain, was the only source of supply. This condition made the water rights and mill the prized possession of every community. Such older water power centers as Ypsilanti, Battle Creek, Grand Rapids, Allegan, South Bend and others are evidences of this influence. Besides these larger centers, every small head on a stream was provided with a mill site where a small amount of power was available. Most of these old mills are still in existence, a few in operation, still others being preserved for their historical interest. A few have been rebuilt into electric plants. There were probably five hundred such small mills in the lower Peninsula of Michigan.

When railroads moved raw products to the market, and finished products from the manufacturing centers on an economic basis which could not be met by the local water power, the value of the water power soon declined. The coming of good roads and the increase in the number of automobiles manufactured also had their effect in permitting transportation and trading over a much larger range. It was in this manner that the small local mill disappeared.



WATER POWER
IN
MICHIGAN

Fig.

Some of these smaller mill powers were taken over and electrified because it seemed uneconomical to abandon them. Electrification of the small mill imposed many handicaps, however. Their use even in systems is limited since such systems have a great deal of electrical capacity and a weak link in such a network is not permissable. For these reasons even the smallest station in an interconnected system must be equipped with the highest grade equipment. This fact was usually prohibitive to further use of many of the small mill powers. In those cases in which this cost was met, there was needed a complete rebuilding of the power house structure, installation of new and expensive machinery and equipment, and the construction of a high voltage station for connection in transmission, was necessary. The expense caused by such an investment, which raised fixed charges to such a figure that generation was no longer profitable, made such an outlay of capital uneconomical. It was for these reasons that many of the smaller mill powers were abandoned with no further use.

The present developed water power in Michigan is perhaps two-thirds of the total capacity which can economically be developed. This conclusion is substantiated by the information presented in graphical form in the accompanying graph. The writer is indebted to Mr. Edward M. Burd, Engineer, Allied Engineers, Inc., for this information which was published in the "Michigan Engineer."

One of the lowest generating costs is shown by the Union Carbide Plant at the Soo, an installation of 40,000 Kilowatts operating continually and with sufficient pondage by virtue of having Lake Superior as a pond. Next in order of economy, and comprising the first twenty-five percent of Michigan's capacity would be the larger plants on the the more favorable sites. This brings the total up to around six mills per kilowatt. All of the plants built since the World War do not produce energy as cheaply as this figure. For these reasons, the best remaining hydro projects for utility operation at the present time entail a total energy cost, based on operation at load center, on a basis comparable to steam power, in excess of seven mills per kilowatt. Estimated total developments now aggregate perhaps 50% and the limit under favorable conditions would seem to be not over one and one-third cents, or 75% of the total. This estimate would leave the last 25% unattainable according to present standards. Or more simply put, two-thirds of Michigan's potential water power is at work and this is by far the best portion.

The upper term of one and one-third already referred to is a relative term, however. That is, there are other factors affecting this value. One of these is competitive power and changing values. It is also quite definitely limited by utility income, as the primary sale net income at the present time of two large systems in Lower Michigan is 1.6 cents. per kilowatt hour. Some margin must also be left between generating cost and the net income to cover allowances for line and distribution losses, to cover the cost of doing business, return to stockholders, and a margin of profit. Primary sale contributes 50% of the whole utility disposal which gives some idea of the relatively small margin which is left. Since the utilities develop 85% of the total water power output in the United States, and perhaps a greater proportion in Michigan, and the rest largely is small powers already described, the economic limitations of any further development for any purpose appears evident.

Interconnection of steam and hydro systems seems to present the maximum of economy in generation and also the best prospects for hydro plant development. It is very important to keep the two forms of energy generation in the proper proportions. Some argue for the development of all hydro for the purpose of conserving our natural resources, national economy, and the exercise of the rights of the public to enjoy these sources of apparently cheap power. Other people propose the use of steam as an alternative since it can be produced more cheaply than hydro power. The opinion of the engineer is that these two systems instead of being rivals, should more properly be classed as partners. It is in this connection that the development of this project is undertaken, and the reasonableness of such interconnection will further be described.

Near Niagara Falls the industrial district centering about Buffalo is supplied with a large amount of steam generated power. On the other hand, it is equally true that large hydro plants are being used in such cities as Pittsburgh, Philadelphia, Baltimore, on interconnected systems, all favored with respect to a cheap supply of coal and cooling water. It is thus seen that the proportion in which to combine steam and hydro is dependent on natural laws and is controlled by elements which enter into the total cost of production.

Furthermore, interconnections are now underway to bring hydro power to New York City and Washington, D. C. These

examples should serve to dispel the usual conclusion that hydro power can be used profitably only when steam is at a distinct disadvantage.

The fundamental difference between the economies of steam and hydro generation is that steam involves a low first cost and high operating cost, while hydro generation is characterized by a high first cost and a lower operating cost. Consequently, a hydro company generally expends a large portion of its gross revenue on bonds and fixed charges. The expenditures may be briefly summarized as follows:

1. The natural factors or conditions affecting construction and operating costs, or what have been called the "characteristics of the site."

2. The use and market characteristics as affecting the sale price and value of the power when developed.

The first item includes geological features as affecting the available foundations for structures, particularly the dam. Topographical conditions are also of great importance in determining the dimensions of the dam, and thus largely affect the cost and the relative proportion of the fall or head to be developed by the dam or by the waterway. Storage possibilities at upstream sites are of special importance.

Operating costs may also be affected by especial conditions which may prevail on a given stream. For example, a stream subject to frequent floods or high-water periods may have the power at the dam site frequently curtailed by backwater, and such a condition will require renewal of flashboards. The presence of ice, particularly anchor ice, on the streams having numerous falls, also introduces troublesome problems of operation.

Under the second item in the above summary, the characteristics of use and market include the conditions particularly affecting the sale price and the value of the generated power.

For example, a factor of vital consideration is the closeness to market. A water power site may be exceedingly low cost development, but situated so far from any possible market as to be out of the question as far as economy is concerned.

The cost of other power at the available site and market is of great importance also as affecting the sale price of the water power.

Load factor as affecting the use ofthe power is of great importance as certain features of the water power development particularly the power house and equipment, vary nearly in inverse proportion as the load factor.

The corresponding elements entering into the cost of steam power are:

1. Fixed charges on plant and transmission costs, including interest and depreciation, taxes, etc.

2. Operating costs, including fuel, labor, maintenance and repairs, and miscellaneous items.

An examination of the two groups of factors given in this comparison shows that the elements of water power cost are practically the same as those of steam cost of generation, except that the fuel outlay is missing as an operating expense. These elements differ greatly in amount, however.

To draw a conclusion, the limit in allowable cost of a water power plant is reached when its fixed charges become so large as to offset the lower operating costs which it boasts. On the other hand, a water power site which can be developed at low cost has a correspondingly high value as a water power privilege, reflecting the advantage in cost of power over that of steam.

A comparison between the two sources of power involves then, a comparison and a proper evaluation of the above elements, namely fixed charges and operating costs.

A low-head development, such as the project considered in this design, has such a large part of its total cost in dam, water rights, etc.,that any additional plant capacity will cost comparatively little, and in fact will doubtless compare favorably with equal steam cost. This may not be true, however, of high-head plants, in which incremental costs, such as the installation of additional water conduits, necessitates additional outlay. Incremental cost is not of such great significance in a low-head development, and as a result, the over development becomes economical; since little is added to the investment. The use of hydro to replace steam or to delay the system of steam construction program, justifies the added investment, because of the saving in operating costs and savings in fixed charges which would be necessary in steam plants.

Since hydro generally costs more per kilowatt hour installed than steam capacity, it might seem possible that hydro could be considered in districts where power can be generated from low fuel cost. But it has been mentioned that saving is the low operating cost of the hydro power. If it can replace the production cost of corresponding steam energy, and at the same time, avoid the investment in steam plants that would be necessary, it is possible under many circumstances to justify the development of water power. There is a decidedly delicate balance between the two factors of operating charges and fixed charges, the appreciation of which has been a recent development in this study.

Whether a hydro plant has the ability to be servicable in replacing steam depends primarily upon the size of the system and the sharpness of the peaks on the load curves, The larger the system, the greater are the possibilities for fitting the available hydro power units into the daily or weekly load curve. Such an arrangement replaces a large amount of steam capacity. In such cases the hydro plant effects a saving in fixed charges.

Another type of service, somewhat more indefinite and harder to evaluate in terms of dollars and cents, is termed the "peak accomodation." For example, there may exist in a given system, a number of old steam systems still kept in service, although uneconomical. These operate but a few hours a day. Some of the difficulties encountered in the operation of such units, and especially their change over to hydro power combinations, have already been mentioned. In such cases, it may be that the run-of-river hydro plants have sufficient pondage to regulate the total weekly pondage requirement. A little less energy may be generated in one part of the week in order to conserve water for the remaining part and permit an uneconomical steam plant to be shut down cold for several consecutive days. This is a great aid in boiler room economy.

Such economy in steam operations appears to be greatly affected by boiler room economies. If sufficient pondage is available, the daily discrepencies in load estimating may be shifted to the hydro plant so that the base load system of steam plant operators know further in advance what load will be expected of them. Also when a hydro plant has been installed in excess of continuous power at the time of minimum stream flow, it is possible by manipulation of pondage to utilize excess capacity to reduce the overall cost of energy.

The fourth service, maintaining system frequency, can be rendered to good advantage by low-head hydro plants. Modern steam plants have a surprising ability to carry load swings but at a disadvantage. Since the water passages are long in a high-head plant, and also because of the danger of water-hammer in the penstocks, these installations are of little assistance. The low-head plant with its short water passages does not suffer this disadvantage making load changes of less consequence.

The trend toward general inter-connection of power systems is particularly favorable for hydro development, for it brings hydro power into a field formerly occupied solely by steam. It gives hydro power the desired conditions to make possible the use of cheap incremental capacity. Interconnection gives the hydro a wider market for off-peak energy that can be disposed of during times of abundant flow.

" Conclusions make it almost obvious that sucessful water power development is dependent upon the opportunity of rendering a substantial service to a widespread market for power which is primarily supplied by steam."*

"Power" September 1929.

CHAPTER 111.

PLANT DESCRIPTION

In the following pages, an attempt is made by the writer to condense into what might be termed an introduction to the design proper, certain of the features of the development which are included in detail in separate sections.

One of the unique features in connection with the provision for adequate spillway capacity, a prime requisite for stability and security of an earth dam, is the semi-permanent flash-board arrangement. The bottom of the spillway channel, over which the water passes, is constructed as a reinforced concrete slab resting upon two transverse walls running parallel to the center line of the dam. Running per-perpendicular to the center line of the dam are cross walls designed to support the pavement slab, since the top of the spillway is made up of a paved highway in the same manner as the remainder of the length of the dam. These slabs are designed to support traffic loads of considerable magnitude as it is expected that the road will be used as an entrance to the power house to be used by heavy trucks laden with equipment. The discharge channel is formed by a slab of eight inch thickness laid on a bed of cinders to assist in the problem of drainage. A tile embedded in the cinders assists in the drainage.

Attention is now directed to the design of the semi-permanent flash-boards.

Flash-boards generally consist of a series of panels supported on pins set in the masonry crest of the dam. The pins are designed to bend over under pressure and loosen the structure when the water in the ponded area reaches a certain elevation. In such an arrangement, the flash-boards are carried down-stream by the flood water and are destroyed. This makes it necessary that before the next period of high water arrives, a new set of flash-boards be procured and installed. This is a crude and expensive method of affording a protection.

For this reason. a type of semi-permanent flash-boards is effected in this development in which much the same type of arrangement employed in the temporary design is used except that provision is made for fastening the flash-boards. The flash-boards are constructed of cypress wood and are bolted together at the top and bottom with supports of the same material. In addition, at the bottom of the flash-boards is placed a steel angle which forms a shoe on which the structure moves. At a point between the two supports determined by the method of moments, a hinge connection is fastened by means of a pin connection . This hinge acts as a pivot around which the flash-boards act. This pivot is so arranged that a rise of water in the ponded area of five feet will cause the flash-boards to collapse and occupy a recess in the bottom of the spillway floor. The steel shoe fastened to the bottom of the boards permits of easy passage across the slab which forms this floor. In order that the structure be stable and not collapse at uncalled-for times, an additional weight is placed at the point of attachment of the shoe angle to care for this emergency. A recess in the timber beam, cast in the floor of the spillway, in the form of a "V" holds the structure when in a vertical position.

This simple arrangement allows for a maximum rise in water level and also affords adequate protection against over topping. It is in addition an inexpensive design and very practical since it is not necessary to replace any of the parts after an occasion of high water has arisen for the use of the spillway.

The power house is here described in two sections; the substructure and the superstructure.

There are two types of substructures in use in the modern day design of power houses. The first makes use of a basement under the structure in addition to the provision for necessary water ways. In the second type the turbine is mounted on a barrel structure which permits the omission of the basement. The first type is chosen for several reasons. In the first place when a basement is provided, the generator is at floor level. Also outside air is available for the cooling of the equipment. A still more important feature of the basement is that it provides a storage place for transformers and other equipment such as electrical conduits and oil and water piping on the ceiling.

The units are placed in a row to facilitate handling by the traveling crane which spans the entire operating floor of the power house. Two 2500 H. P. turbines are placed thirty two feet apart and a 5000 H. P. turbine is placed at thirty two feet from the center line of the other turbine. These turbines are directly excited by generators on the same shaft. The generators are on operating floor level at elevation 280 feet. A stairway provides access to the auxiliary floor at elevation 271 feet. The Francis Vertical Plate Steel Cased turbines are in installed at floor level 259 feet. The penstock also enters the substructure at the same elevation. These details may be referred to in the section on the design of the power house.

The floors of the power house are constructed of reinforced concrete, of the slab construction. The width of the power house is 60 feet. This entire width is spanned by the travel of the 50 T. crane and to facilitate handling of generators, transformers and turbines, sections of floor five feet in width and almost the entire length of the power house, namely, 75 feet, are removable. These sections are constructed of Blaw-Knox steel grating. This grating is constructed of flat bearing bars crossed at right angles by twisted cross bars, the intersection being made by one-piece electroforging under enormous pressure, This is done without cutting, slotting or punching any of the bars or removal of metal. This grating as employed in this design to take the place of a thoroughly reinforced concrete slab gives equal strength and rigidity. A special design is necessary to support this grating at the places where the generator shaft projects above the elevation of the operating floor. This design will be furnished by the company. The design of the support and the reinforcement of concrete slab adjacent is covered in the section under "Floor Slabs."

The substructure walls are constructed of reinforced concrete. A special feature made necessary by the integral construction of the dam and power house, is the retaining wall as foundation for the superstructure as shown on the detail plans. This wall is designed to resist an unusual combination of loads, namely the load of the superstructure, or the superimposed load; the load or pressure of the earth fill forming part of the dam; and lastly its own weight. The wall is constructed as a counterforted wall. The toe projects a distance determined by design requirements, under the floor which is the lowest floor under the upstream half of the power house. The thickness of this floor is increased considerably to allow for the application of indeterminate loads. The foundation under the retaining wall are made as supported on the same subsoil as the dam itself, namely the mudstone layers later referred to.

The turbine setting is described on drawing p.65, the sectional view of the substructure. The entire weight of the turbine is carried through the barrel and the turbine speed rings to the foundation.

The recommendations for draft tube design are usually furnished by the manufacturer who fixes the requirements. The draft tube position is located on the section view, however, in its probable position.

The electrical cables and conduits are placed on the ceiling of the transformer room.

The ventilation of the generators is a very important consideration. The openings on the substructure of the downstream foundation wall of the power house will provide ample circulation of cool air to the equipment.

A shower room is also provided in the basement of the power house, in which room are provided also lockers and toilet facilities.

The primary function of the superstructure is to support the machinery and also to provide facilities for handling such machinery and equipment.

The clearance of the crane, namely 7 feet 10 inches, fixes the height of the power house. In this design the height is chosen to be approximately 20 feet from the operating floor at elevation 280 feet to the bottom of the roof truss. The roof truss is of the Fink type and spans about 60 feet with trusses at fifteen feet center to center. The roof covering consists of corrugated steel sheets, supported directly on the purlins. A lining made of two layers of felt and two layers of tar paper is placed directly on wire netting stretched over the purlins. This roof construction prevents condensation under the metal roofing and also acts as an insulator. The design of the roof truss is covered in detail in a later section.

The columns in the superstructure are built of built up sections. The design of these columns is made particularly difficult since the application of the crane loads at its upper third length causes a marked tendency to bend. This load is very eccentric and causes further complications.

The exterior columns are brick veneered on the outside and Natco-Vitritile covered on the inside. This type of tile installation is an extremly attractive finished face glazed structural fire clay tile. It is used throughout the power house both for interior walls and also partitions. The tiles are carried over window openings by steel rods of specified size embedded in concrete, which is poured in the openings in the tile.

The floors of the power house are constructed of reinforced concrete construction slab and beam design. The design is entirely orthodox in all cases and is of special type in the installation of the steel grating already referred to.

The floors are to receive a dust preventive application.

Doors and windows are included in the separate section of power house details. Both are of Truscon manufacture.

Lighting of the power house is shown on the drawing . The type of fixture chosen for illumination of the operating floor is the dome reflector type. These fixtures are fastened at indicated intervals on the lower member of the roof truss. These fixtures disperse light in all directions and allow a minimum to pass upward into the unused portion of the roof constructed of the open members unenclosed.

The following is a list of the equipment for which space is provided in the superstructure:

Main generating machinery;

Turbine machinery, governors, pumps, and tanks;

Motor- generator sets;

Compressed-air equipment;

Water-supply pumps;

Switchboard and low-tension switches and buses;

Storage batteries;

Transformers, oil tanks with filter and necessary pumps;

Telephone equipment;

Lavatory;

Office.

Should space become limited in the superstructure, ample room is available in the substructure, and some of the above equipment may as well be placed in the basement.

CHAPTER IV.

HEAD WATER CONTROL AND ACCESSORIES

The main purposes for which stream flow measurements are necessary are as follows:

(1) To estimate the average annual energy output of the development;

(2) To estimate the additional energy provided by a proposed storage reservoir;

(3) To estimate the minimum annual energy output;

(4) To determine the capacity of a storage reservoir to equalize the flow during the period to a minimum;

(5) To estimate the minimum daily output without storage.

Considerable time was spent in making the determinations relative to stream flow measurements and estimates since accurate measurements have not been taken on the Au Sable River, the river which was used as a comparison since the location of the proposed dam and power plant were not known. This obviously precluded that many of the results as far as maximum and minimum flows be largely estimates, with the possibility that they would be largely erroneous due to the fact that they were obtained by the method of comparison. The results of this part of the investigation, namely the comparison of the two rivers, have been included in graphical form in the accompanying sheets.

Low flow is assumed to be at the stage shown on the map of the site contained in the pocket in the back of this the-

sis, and represents a discharge of about 800 second feet continually. This corresponds to a surface elevation of 243.5 feet at the boring base line. It is assumed with some basis from past experience that no greater rise than 3 feet would take place except during the Spring break-up in April, at which time the probable rise would not exceed 8 feet.

The river flow assumptions given here were obtained through the courtesy of the United States Department of Agriculture, Division of Stream Flow Measurement, State Building, Lansing, Michigan.

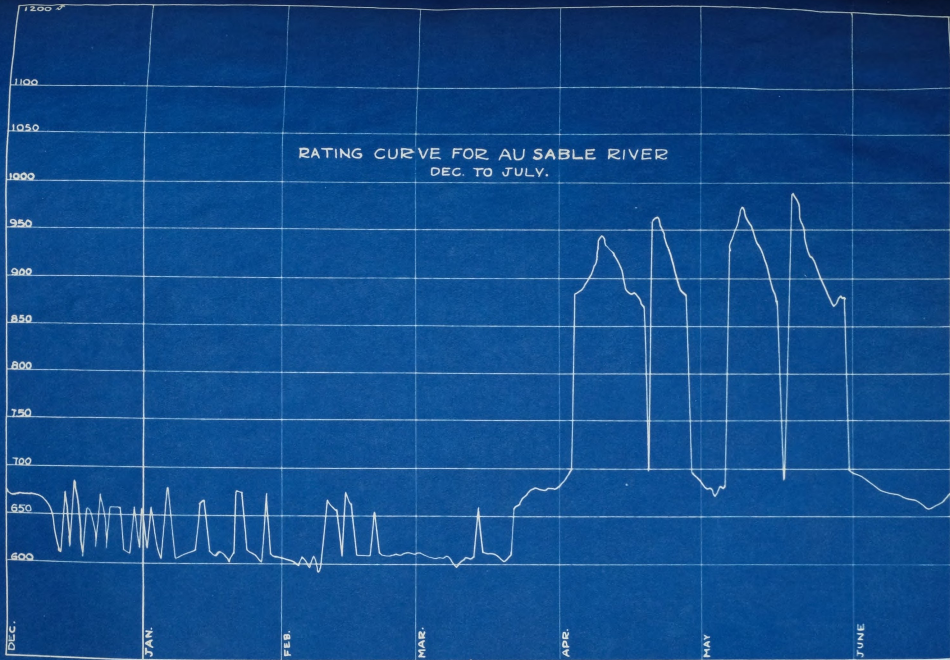
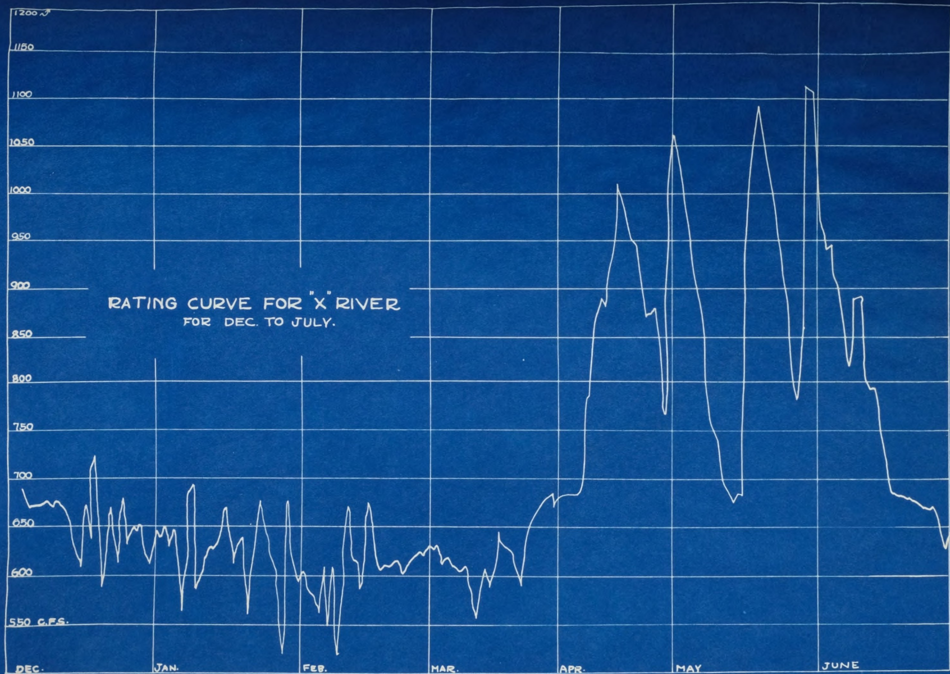

RATING CURVE FOR AU SABLE RIVER
DEC. TO JULY.
1200 ft
1100
1050
1000
950
900
850
800
750
700
650
600
DEC.
JAN.
FEB.
MAR.
APR.
MAY
JUNE

Fig. 2.


RATING CURVE FOR "X" RIVER
FOR DEC. TO JULY.
1200
1150
1100
1050
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950
900
850
800
750
700
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600
550 C.F.S.
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APR.
MAY
JUNE

Fig. 3.

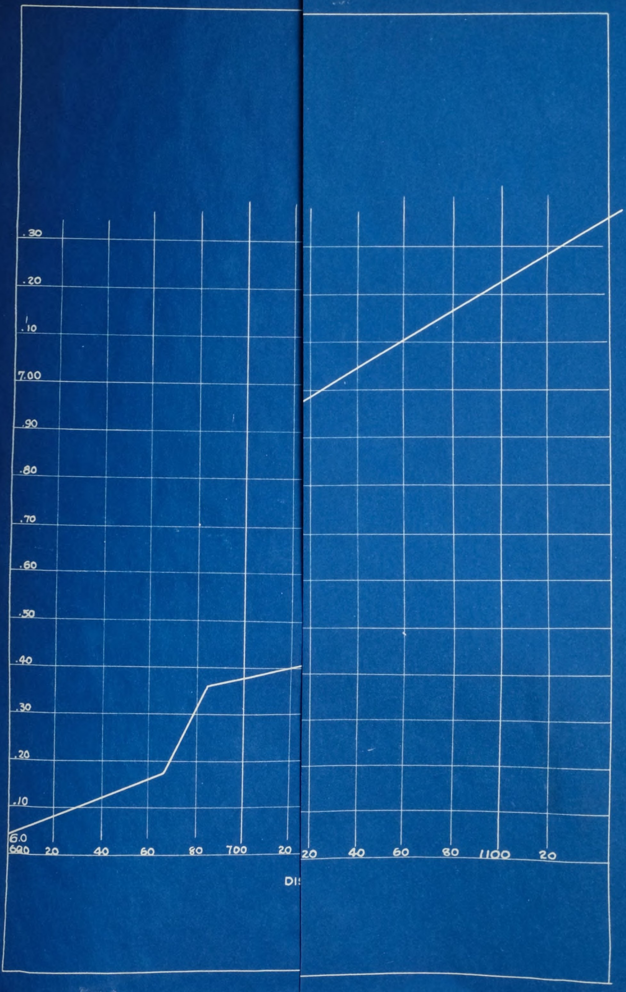

.30
.20
.10
7.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
6.0
620
20
40
60
80
700
20
20
40
60
80
1100
20
DI

Fig. 4.

PART A

Guage-height record for the Au Sable River near Red Oak, Mich.

Date	"X'" Dec.	Jan.	Feb.	Mar.	April	May	June
1.	----	6.13	6.04-	6.09	6.32	6.42	6.49
2.	----	6.11	6.05	6.10	6.36	6.37	6.48
3.	----	6.14	6.00	6.06	6.47	6.35	6.48
4.	6.36	6.09	6.00	6.08	6.47	6.20	6.45
5.	6.20	6.13	5.96	----	6.57	6.35	6.40
6.	6.19	6.08	6.06	6.05	6.60	6.31	6.36
7.	6.24	5.97	5.91	6.04	6.58	6.80	6.34
8.	6.22	6.32	6.06	6.06	6.68	7.00	6.30
9.	6.22	6.32	5.87	6.03	6.72	7.11	6.30
10.	6.21	6.00	6.10	5.95	6.96	7.20	6.28
11.	6.23	6.04	6.19	6.00	6.93	7.11	6.26
12.	6.22	6.06	6.07	6.06	6.85	7.02	6.21
13.	6.17	6.09	6.07	6.10	6.79	6.91	6.20
14.	6.12	6.09	5.97	6.04	6.77	6.79	6.17
15.	6.07	6.13	6.27	6.13	6.65	6.65	6.13
16.	6.06	6.20	6.15	6.10	6.57	6.59	6.09
17.	6.26	6.10	6.06	6.09	6.57	6.51	6.13
18.	6.11	6.05	6.05	6.09	6.58	6.46	6.09
19.	6.40	6.11	6.06	6.09	6.58	7.27	6.13
20.	6.12	6.10	6.06	6.10	6.45	7.24	6.14
21.	6.00	5.94	6.07	6.10	7.05	7.03	6.16
22.	6.19	6.07	6.08	6.12	7.11	6.82	6.23
23.	6.12	6.26	6.04	6.16	7.04	6.75	6.26
24.	6.06	6.16	6.06	6.24	6.88	6.77	6.25
25.	6.27	6.[illegible]	6.08	6.27	6.78	6.67	6.19
26.	6.10	6.06	6.08	6.34	6.67	6.60	6.15
27.	6.13	6.05	6.09	6.20	6.60	6.54	6.22
28.	6.13	5.84	6.10	6.28	6.57	6.51	6.88
29.	6.14	6.26	----	6.31	6.45	6.60	7.32
30.	6.08	6.06	----	6.33	6.43	6.56	7.00
31.	6.06	6.03	----	6.32	----	6.58	----

Additional Data:

Au Sable near Red Oak Michigan between Sec.2-3. T.26N. R 1E. One-half mile South of Red Oak Post Office Oscoda County. Four miles North of Luzene.

PART B

Date	Width	Area sq.ft.	Vel.	"X" Guage height	"Q" Q-'ft.	Method	Coeff.	Time	Fact.
Dec.4	72.4'	261	2.62	6.36	685	.2-.8	17	$1\frac{1}{3}$	107.8
Jan.8	70.0'	246	2.72	6.18	668	do	33	$2\frac{1}{12}$	108.0
Mar.12	69.5'	228	2.67	6.06	609	do	33	$1\frac{7}{12}$	100.5
Apr.7	72.5'	269	3.30	6.59	887	do	35	$1\frac{1}{2}$	134.5
May27	72.5'	277	3.26	6.64	903	do	36	$1\frac{1}{2}$	136.0

Computations for obtaining values for "Q" for data on guage readings given in Part A:

Data given in Part A are furnished through the courtesy of the United Statees Department of Agriculture, Division of the Interior, and represents information taken during 1931. The information given in Part B is a partial determination of the data shown in the first part. From the sample computation shown below, the relationship between A. and B. is readily seen. This simple computation obviously does not give what might be called accurate results for the river in this problem, but for the purpose of establishing assumptions regarding maximum and minimum discharges and for subsequent calculations, it is entirely satisfactory.

Sample Computation:

Equation: $x : Q = x' : Q'$

$$6.18 : 668 = 6.36 : Q'$$

$$Q' = 637 \text{ c.f.s.}$$

Code:

x' = guage reading in Part A.

Q' - discharge in Part A.

x = guage reading in Part B.

Q - discharge in Part B.

TABLE OF DISCHARGE

Date	Dec.'30	Jan.'31	Feb.'31	Mar.'31	Apr.'31	May'31	June'31
1	685.0	661.0	608.0	613.0	681.0	691.0	700.0
2	685.0	615.0	609.0	614.0	685.0	687.0	698.0
3	687.0	661.0	60410	610.0	697.0	784.0	698.0
4	685.0	613.0	604.0	612.0	697.0	670.0	695.0
5	670.0	661.0	600.0	-----	885.0	684.0	690.0
6	669.0	612.0	610.0	609.0	885.0	680.0	685.0
7	674.0	602.0	595.0	608.0	886.0	926.0	682.0
8	673.0	680.0	610.0	610.0	910.0	952.0	678.0
9	671.0	665.0	59010	607.0	916.0	967.0	678.0
10	673.0	604.0	614.0	598.0	946.0	980.0	677.0
11	672.0	608.0	669.0	604.0	943.0	966.0	676.0
12	667.0	610.0	658.0	610.0	942.0	955.0	670.0
13	660.0	613.0	658.0	605.0	925.0	940.0	670.0
14	611.0	613.0	602.0	608.0	920.0	925.0	668.0
15	610.0	661.0	676.0	662.0	905.0	905.0	670.0
16	676.0	670.0	664.0	614.0	885.0	887.0	667.0
17	615.0	614.0	610.0	613.0	885.0	875.0	662.0
18	690.0	608.0	609.0	613.0	886.0	695.0	613.0
19	660.0	615.0	610.0	617.0	871.0	880.0	662.0
20	604.0	614.0	658.0	615.0	695.0	990.0	663.0
21	658.0	598.0	612.0	614.0	960.0	986.0	665.0
22	660.0	610.0	608.0	661.0	966.0	955.0	674.0
23	610.0	676.0	610.0	665.0	958.0	942.0	677.0
24	676.0	675.0	610.0	673.0	937.0	920.0	675.0
25	614.0	675.0	612.0	676.0	933.0	922.0	668.0
26	661.0	610.0	613.0	684.0	907.0	907.0	664.0
27	661.0	609.0	612.0	620.0	886.0	886.0	671.0
28	612.0	587.0	614.0	678.0	885.0	880.0	[illegible]35.0
29	610.0	676.0	-----	680.0	885.0	975.0	995.0
30	612.0	610.0	-----	682.0	695.0	887.0	952.0
31	610.0	607.0	-----	681.0	692.0	882.0	-----

The variation in the discharge for successive days, as will be noted from an examination of this table of discharges, is due to a change in factor from the data furnished by the stream measurement department of the United States Dept. of Agriculture. The actual change in the discharge would not be so noticable because the width is the only variable of any account. See part B.

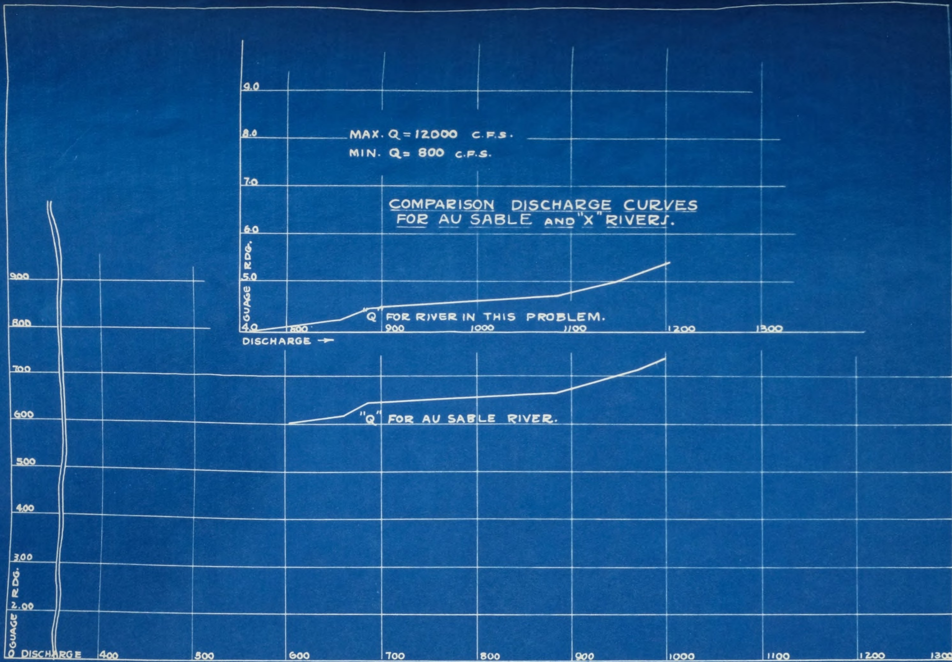

COMPARISON DISCHARGE CURVES
FOR AU SABLE AND "X" RIVERS.
MAX. Q = 12000 C.F.S.
MIN. Q = 800 C.F.S.
"Q" FOR RIVER IN THIS PROBLEM.
"Q" FOR AU SABLE RIVER.
GUAGE RDG.
DISCHARGE →
9.0
8.0
7.0
6.0
5.0
4.0
800
900
1000
1100
1200
1300
900
800
700
600
500
400
300
2.00
0
DISCHARGE
400
500
600
700
800
900
1000
1100
1200
1300

Fig. 5.

Pondage is defined as the holding back and releasing later of water at a dam of a water power development, (1) to equalize daily or weekly fluctuations in river flow, (2) to permit irregular hourly use of water by the wheels to accord with the fluctuations in load demand. In order to make a rather accurate and quick determination it is noted that 43,560 cubic feet will provide a flow of approximately 1 second foot. In order to make a similar comparison, it is necessary to measure the volume which can be considered to furnish pondage.

For purposes of computation, the Western boundary comprising the water shed is assumed to be located .74 miles west of the most northerly bank of the river as is shown on the Location Map; the Northern boundary as being the top margin of the map; the other boundary being the irregular line formed by the center line of the dam and contour line 270 ft. This area was planimetered, that is the area between successive contours was planimetered and these areas multiplied by the average height or difference in elevation and the contour interval. This area as planimetered was equal to 5,782,960 square feet. This method proved a rather unsatisfactory one and a second one was devised and finally dopted.

The method used consists in dividing the entire area enclosed in the boundaries mentioned into 100 square foot areas ans then estimating the corner elevations from contour lines which were already drawn and then making the necessary computations. This method seemed entirely suitable and was more practical since the contours were so widely distributed over the area.

The results are included in the next few pages. The approximate pondage as determined in this manner was 194,413,250 cubic feet. Now referring to the statement that 43,560 cubic feet will provide a flow of one second foot, it is seen that the pondage available at this site will only provide water for low water conditi ns, and then only as an equalizer of daily fluctuations in river flow and also will provide for irregular hourly use of water by the wheels.

These results would have ordinarily been disappointing to promoters of a proposed development, but in this case, not only because this problem is entirely theoretical, but also because the boundaries of the water shed were arbitrarily chosen, did the results seem at all plausible.

After the dividing of the entire area into 100 foot square areas, the elevations at the corner points were determined from existing contour points and by interpolating in the cases that a contour line did not run close enough to the corner in question.

The next step was to refer all these elevations to a single plane to figure the volume. The plane used was, of course, the top of the proposed dam at elevation 301 feet. A table was constructed, a portion of which is included below to illustrate the procedure, in order to facilitate the work. All the possible corner elevations were listed in one column and opposite each of these, the difference in elevation which such a height would be referred to the datum, was listed. Then the complete map was covered and the differencces in elevations recorded.

In accord with methods already established, the corners

that appeared in one square only were multiplied by 1; those that appeared in two squares, by 2; those that appeared in three squares, by 3, and so on. The total was divided by 4 to place the result in cubic measurement.

Example:

Elevation	Difference in elevation between corner and datum.
244.1	56.9
244.2	56.8
244.3	56.7

Computation:

$$(M \times 1) + (M \times 2) + (M \times 4) = \frac{77,565.3 \times 10,000}{4}$$

$$= 194,413,250 \text{ cu. ft.}$$

A large number of borings were taken at the site of the proposed dam by the Allied Engineers and a generalization of the foundation conditions encountered is given below.

The general foundation condition is glacial drift for an unknown depth of at least several hundred feet. The top portion represents an old out-wash plain, through which the river has subsequently cut. This out-wash plain was formed by glacial outflows of varying intensity and velocity, some times being very sluggish streams because of back water conditions; and again at other times being very rapidly flowing streams which did, of course, all the depositing and cutting. When sluggish, the stream seemed to have deposited very fine rock flour, geologically known as mudstone, and locally called mudstone clay beds, being very dense, impervious and tough material, which forms excellent foundation and is eroded very slowly by the river flow. These rivers cut through these mudstone layers very slowly so that the erosion can hardly be detected, being as a matter of fact, a cutting of a foot or two over a period of perhaps ten to fifteen years.

During the time when the outflow was rapid, sand and gravel were deposited in layers of considerable thickness, extending below the river valley, and with top at site of dam at elevation 240 feet. Beneath this is coarse sand and gravel heavily water bearing. Above this is a strata of miscellaneous sand and gravel perhaps 25 feet in average thickness, through which the river has subsequently cut its present course. Next above this layer is another layer of mudstone averaging 15 feet in thickness, and outcropping in the cut banks where the river has entirely worn through it in ages gône by. Next above this is a miscellaneous deposition of sand and gravel which extends clear to the sand hills.

Experience has shown that it is entirely safe to build upon these mudstone layers that have been referred to and also that they are sufficiently continuous and impervious to form dependable water cut-offs. The intervening layers have been found to be unstable, and protection against any head of water is afforded by steel sheet piling as shown on detail A. This protection will safely shut off the head in question and as an additional protection, the upstream side of the dam is paved for a distance of 25 feet from the top of the dam to the core wall as shown in detail.

CHAPTER V.

THE DESIGN OF DAM; CORE WALL AND APPURTENANCES

An earth dam was once looked upon by many engineers as merely a fill of earth, and its design was not often considered to be worthy of much consideration, and still more often its construction received little supervision; but today, failures that seemed avoidable have directed attention to it as an engineering structure. It is now generally recognized that the careful attention to details is just as important in the design of an earth dam as in any other structure. If properly designed and built, it is not only a safe, but also a permanent and natural structure which blends into the side hills and forms a part of the general landscape.

it is interesting to note the change in attitude which authors of text books on modern hydro-electric practice have taken with respect to the applicability of the earth dam in large installations. Books published two decades ago devote only a small section to the design and construction of earth dams, while the latest text books available present the subject in some length.

A careful study of conditions shows that if attention is paid to such features as adequate spillway capacity; confining the line of saturation to lie well within the downstream toe; providing correct slopes to both the upstream and downstream faces; caring for drainage to prevent free passage of water from the upstream to the downstream slope

causing ulitmate failure; and lastly having freeboard such that there is no danger of overtopping from wave action -- then and only then will an earth dam prove permanent.

It is with these criteria in mind that the design of the earth dam, included in the next few pages and with details included on accompanying drawings, is carried out.

The cross section of the dam is seen to be trapezoidal in shape which is the natural form of an earth bank. A study of the requirements indicated that top width sufficient to carry a paved highway, the design of which is discussed later, would prove to be economical since it fitted in the existing network of roads.

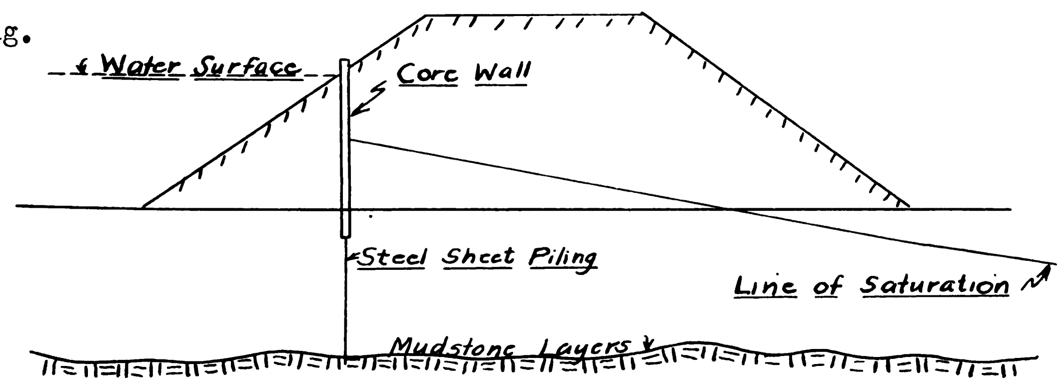
The height to which waves will ride up on the embankment, although not given to exact determination, may be found by making use of an equation for height of waves as follows.

$$* \; H = 1.5\sqrt{D} - 2.5\sqrt[4]{D}$$

where H is the height of the waves, from trough to crest, in feet; and D is equal to the exposure or fetch in miles. Since the actual heights of the waves from mean water is approximately one-half that given in the equation by * Stephenson, it seemed reasonable to assume that a top width of 20 feet, paved with an 18 foot pavement would be adequate protection against overtopping. The design of the spillway and its relation to freeboard allowance will be discussed later.

A slope of 1 on 4 is chosen for the upstream face running from ground elevation of 240 feet up to elevation of the top of the dam, 301 feet, making the total height equal to 61 feet. The slope of the upstream face is picked not so much as a theoretical determination, but is based upon the practical experience of engineers in construction and from their use of structures of this type. This slope of 1 on 2 of the downstream face of the damruns from an elevation 280 feet where a 10 foot gravel road forms the western approach to the power house entrance. This same slope continuous 1 on 2 up to the top of the dam at elevation 301 feet.

The sketch below shows the advantage which is gained by placing the core wall upstream from the center of the dam. This is in order to lower the line of saturation, and the portion of the wall extending above the intersection with the upstream face may also be used as a protection to the slope. This removes to a great extent the danger of washing at this point which often results in failure due to piping.



The core wall is constructed of 1:2;4 concrete reinforced with .5 of 1% reinforcing steel, $\frac{1}{2}$ Ø bars @ 2" intervals near the face of the wall. The core wall is constructed in sections , the first being 25 feet in height and 24 inches in thickness; the second section being of the same height but 20 inches in thickness, the 2 inch inset on both sides for the placing of forms in the process of construction, and the last section 17 feet in height and 12 inches in thickness.

The reinforcement of the top section is further taken care of by allowing the reinforcement of the slab forming the upstream face to extend down and become imbedded in the core wall. This reinforcement is in the form of fabric of electrically welded steel as shown on the detail of the core wall (detail B). The core wall is reinforced in this manner as it is the intention that the two-way reinforcement would provide a sort of a slab action across the areas of unequal pressure and would allow for unequal settlement of the embankment and still remain intact. This form of diafragm core wall is of especial importance in this particular dam because of the relatively heavy load on the embankment caused by the pavement and resulting traffic.

It was stated in the discussion of foundation conditions that although the mudstone layers were sufficiently continuous and impervious to form dependable water cut-offs, the intervening layer, extending perhaps 50 feet below the original ground level had to be blocked off against any head of water. To form this cut-off, sheet piling is driven to a depth of 50 feet and some distance back into the foot hills as is shown on detail A.

The dam in constructed by the method of rolling. Experience has shown that when the embankment material is spread in thin layers and rolled, the effect of settling is relatively small and extends over a period perhaps of years. The layers are made generally of less than one foot, a thickness of six inches being desired in all cases for the material found in the foot hills from which the embankment material is obtained. The material is conveyed to and dumped upon the dam by means of wagons and scrapers are then used to spread the material to a depth of six inches. A watering wagon is used to wet each layer subsequent to rolling and the placing of a new layer. A relatively light roller is employed with a pressure of 50 pounds and a penetration of one inch to the wheel. Hand tamping is necessary close to the core wall which is built as the dam progresses. At the core wall the layers of ambankment material were reduced to three inches rather than six inches as otherwise. A two percent allowance is allowed for shrinkage in the entire structure.

CHAPTER V1

DESIGN OF SPILLWAY AND FLASH-BOARDS

In sufficient spillway capacity is the most common cause of failure of earth dams,and one of the failures that caused great loss of life and property, the Johnstown, Pa., flood, was due to this cause.

A masonry dam can usually withstand overtopping by water without serious damage, but should an earth dam overtop, the washing away of channels and carrying away of the earth from the top and slopes, would be certain to result in its failure. For these reasons much study is given to the spillway design in this development.

The spillway is not intended to be used except a very small portion of the time, only in case of a break in a dam upstream, and in rare instances to discharge excess water not utilized by the turbines. The advantage of the spillway here designed is that it allows for a ponding up of nearly 5 feet additional water over the entire area before discharge takes place.

The plan and details of the spillway are given on drawing B, but to add to its clarity, a discussion of the construction is given at this point.

There are a number of flash-boards which are in common use for the purpose of controlling the head water and the type to use on a particular development is dependent on many factors. In this particular type of development, namely, ome in which water is very valuable because of the very small pondage capacity available, it is advisable to adopt a design

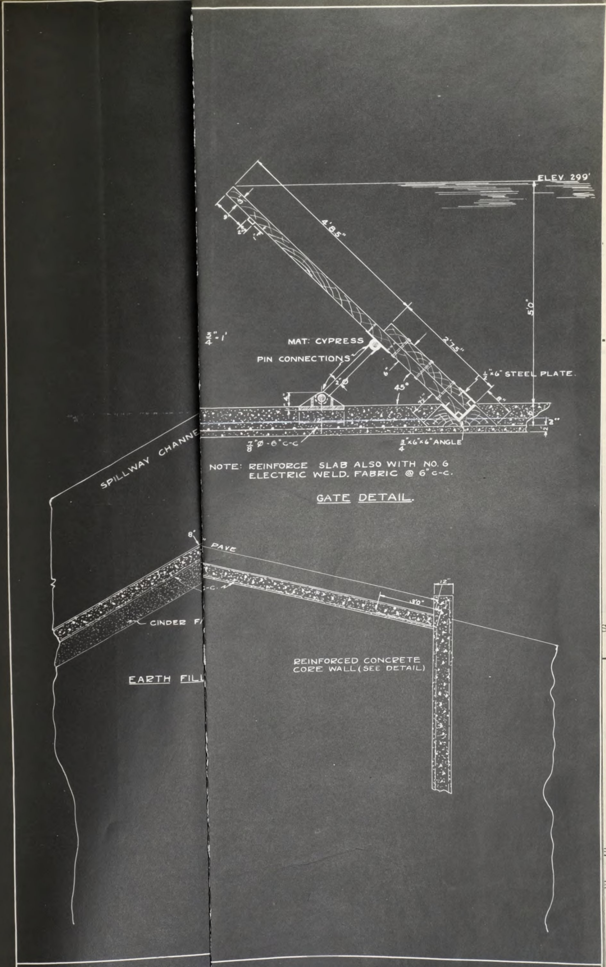


PLATE B. SPILLWAY DETAIL. Fig. 7

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aped

of a spillway which will allow for a minimum of leakage during periods of drought and still allow an adequate protection.

A general description of the flash-boards has already been given in the section entitled "Plant Description", but the semi-permanent device will be referred to in this chapter in some detail in order to illustrate the method of operation.

The flash-boards are constructed of 3" x 3" x 7' pieces of cypress wood, such wood being chosen because of its resistance to decay from direct contact with the elements. These pieces are bolted together at the top with a support formed by a 1" x 2" cross bar of the same material. At a point 2'-7.5" from the bottom end of the panel, the hinge is fastened and this is is turn fastened to a 2" round steel bar which forms the moment arm of the flash-board. In order that the structure be stable and not collapse at uncalled-for times, an additional weight is calculated by the method of moments to care for this emergency. It is found that pieces of cypress of the same dimension but of length 2'-7.5" will cause a condition of stability. This makes the bottom end of the flash-board structure 6 inches in width. On to this bottom end is fastened a 6" x 6" x 3/4" steel angle. This angle is placed at this section to provide a smoother passage of the bottom end of the flash-board structure over the concrete slab of the floor of the spillway, at times when the increase in water level causes it to operate. A recess in the form of a "V" shaped notch is cut out of cypress timber and runs parallel to the center line of the dam. This timber is set in the floor of the spillway at the time the slab is poured. It is in this notch that the angle or shoe of the flash-board sets when the the boards are in their natural position. The fact that the surface of contact between the shoe and the timber is one of the best to minimize the force of friction, aids in the im - mediate functioning of the device when needed, and the weight of the angle aids to cause stability in periods when the lev- el of the water is below that required to operate the flash-boards.

The space covered by the individual panels of the spill- way is 9 feet in length and there are sixteen of these pan- els in the entire length. There are nine flash-boards in each panel and they are placed as shown in the detail on drawing D. The surfaces of contact between the end concrete and the end flash-board are caulked with ashes to prevent leakage, and to further remove this danger, it is specified that the joint be carefully tested during construction, and also that a periodic inspection be made of this detail by the operators in charge of the station.

To illustrate the operation of the flash-board, suppose the water level rises from elevation 294 feet to elevation 299 feet. The weight of the water would be just sufficient to cause the pressure on the upper end of the flash-board to be increased to the extent that this end would begin to lower. Since the entire panel is pivoted, this would cause the low- er end of the structure, the end on which the shoe is fasten- ed to become disengaged from the notch in which it is placed. As the water rushes over the collapsing structure, the angle will slide along the recess provided in the floor of the spillway, allowing smooth , unobstructed passage of the water over the floor of the spillway. At the completion of the period of flow, or after the level of the water has once more lowered to elevation 294 feet, it is necessary to once more raise the flash-boards by panels, placing the angle back in the notch .The structure is now ready again for the next advent of high water.

This simple structure allows for an adequate protection against overtopping and also is inexpensive and practical since it permits operation without any need for replacement of any parts as is the case in the temporary flash-boards.

The construction of the masonry spillway seems to be adequately described by the drawings on Plate B. Suffice it to give a short description of the general method of design and construction.

The spillway is constructed so that a smooth grade is maintained the entire length of the dam, in order to avoid changes in the grade of the pavement. The bottom of the spillway channel over which the water passes, is constructed as a reinforced concrete slab, 12" in thickness, resting upom two cross walls which run parallel to the dam. This slab is reinforced with 7/8 inch round bars, placed at 6" intervals, amd also with No. 6 Electric Welded Steel Fabric at 6" intervals. The cross walls, on which the floor rests, are constructed of reinforced concrete, and are 1' x 2' in cross section, and run the entire length of the spillway. Their reinforcement consists of the same size fabric and steel bars which in turn are linked with the reinforcement of the slab forming the upstream face of the dam in one case, and with the floor of the discharge channel on the downstream side of the dam. These details will be clear on inspection of the drawings already referred to.

Running perpendicular to the center line of the dam, and at 10 foot intervals, are transverse walls which are designed to support the pavement slab. These walls are designed to support traffic loads of considerable magnitude as it is expected that they will serve as an entrance to the power house to be used by trucks heavily laden with equipment. These walls are 12" in thickness and 20 feet in length and 7 feet high. The reinforcement consists of both bars and electric welded steel fabric. This reinforcement is shown on detail D.

The discharge channel is formed by a channel slab 8" in thickness formed by a concrete slab which is laid on a bed of cinders, one foot in thickness. This bed is to care for the drainage of the channel and possible seepage of the water under the channel floor. The slab is completely reinforced with No. 2 and 8 electric welded steel fabric spaced 2" and 16". The sides of the channel are also constructed of reinforced concrete, being walls 7 feet in height, and 8 inches in thickness. The channel narrows from a width of 160 feet at the spillway elevation of 294 feet down to 70 feet at elevation 272 feet. It then runs for 49 feet at the same width of 70 feet. The sides of the channel slope at a rate of 4 to 7. This feature is designed as being an easy and economical section for the flow of water as determined by experiments with the flow of water in open channels. A cross section of the channel is shown on Plate C.

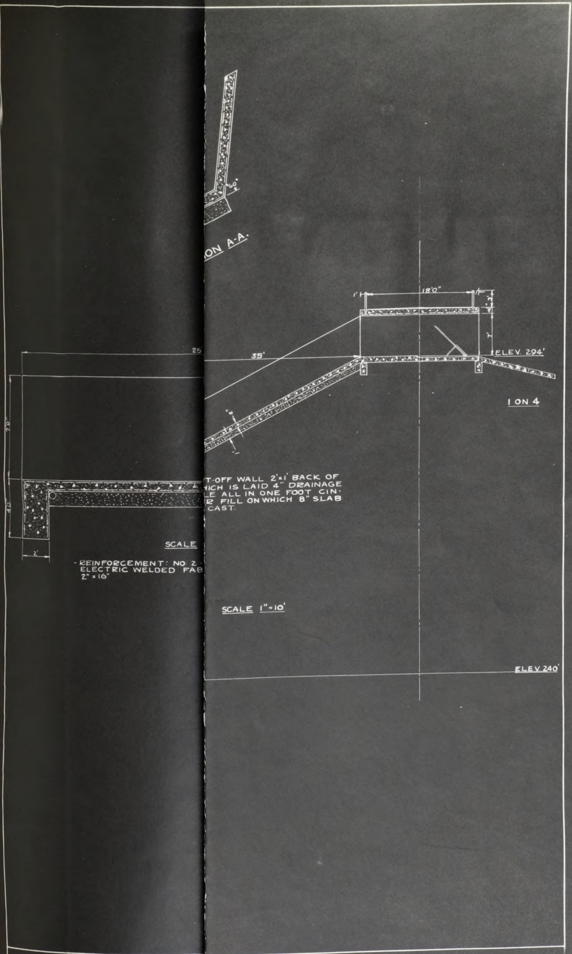


PLATE C. DOWNSTREAM - SPILLWAY. *Fig. 8.*

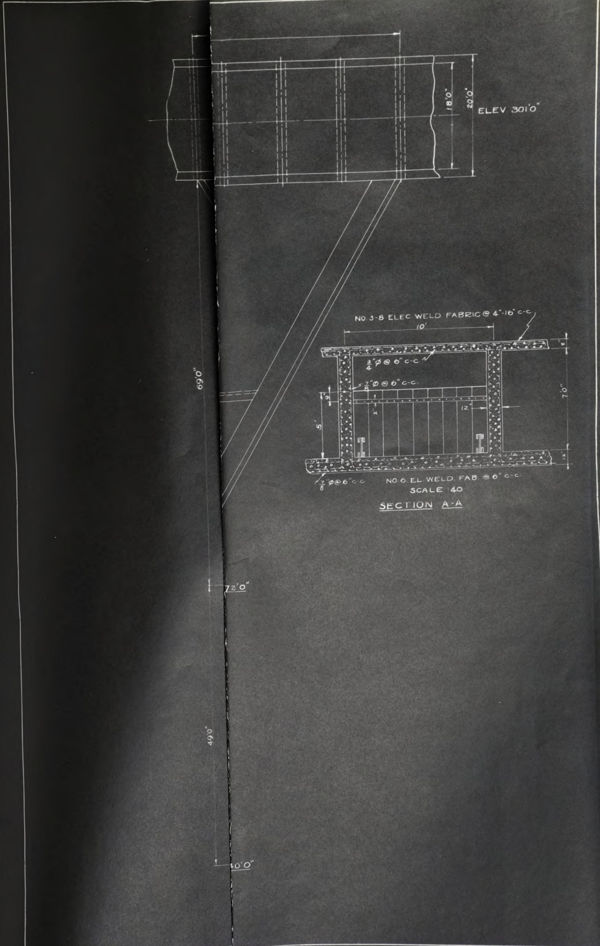
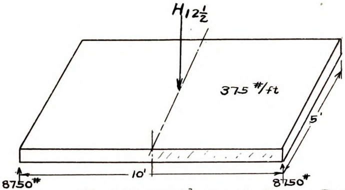


PLATE D.

SPILLWAY PLAN & DETAIL Fig. 9

Drainage of the channel is cared for by a 6 inch tile embedded in cinders and running from the slope from elevation 301 feet to elevation of the original ground elevation at 240 feet. In addition, there are provided three installations of 4 inch tiles placed behind cut-off walls, shown in plan and detail on Plate C.

The design of spillway floor slab:



Loadings: Standard $12\frac{1}{2}$ truck load. This load is equivalent to a total load on each traffic lane composed of a uniform load of 375 pounds per limeal foot and a single concentrated load of 17500 pounds. These loads are in accordance with specifications issued by the Michigan State Highway Department and are in use by that department in the design of pavement slabs.

Moments: (a) due to uniform load of 375#/lin. ft. =

$$375 \times 5 \times 5 - 375 \times 5 \times \frac{5}{2} = 4688 \text{ pound ft,}$$

(b) due to concentrated load of 17500# =

$$8750 \times 5 = 43750 \text{ pound feet.}$$

Then: Total moment due to both loads = 48440 # ft.

Also $bd^2 = \frac{M}{K}$ $n = 12$; $f_c = 1000\#$; $f_s = 20000\#$

Use 2500 pound concrete; $p = .0094$

$$d^2 = \frac{48440 \times 12}{5 \times 12 \times 164} = 59''$$

Or $d = 7.7$ inches or 8".

Make the slab 10 inches in thickness to care of protective coating for reinforcement.

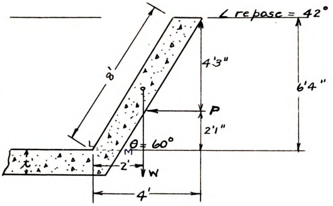
And $p = .0094 \times 12 \times 8 = .903$ inches

For this reinforcement use $\frac{3}{4}$" round bars @ 6" c-c. This will give an area of .88 square inches with the elec-

tric fabric in the form of No. 6 electric welded steel fabric at 4" horizontal and 16" vertical spacing will give the required area of steel to care for requirements of design.

<u>The design of the spillway channel</u>:

The design of the spillway channel shown in detail on Plate C. is here described as a method of design in accordance with the principles of retaining wall design for a wall to resist the pressure of earth surcharge. The reference to the formula used may be found in "The Design of Masonry and Foundations", by Williams om page 259.



(a) $P = \frac{1}{2}\ w\ h^2\ k_o$

$P = \frac{1}{2}\ 110\ x\ (6\frac{1}{3})^2\ x\ .05$

$P = 110$ pounds.

(b) Summation of moments around M = 0

110 x 2' 1" = 2 x 8 x 1 x 150 x t'

t' = .095 inches.

This thickness is obviously too small and merly indicates that the requirement for moment is not the governing factor in the design. For this reason use an eight inch slab as shown.

(c) Investigation for shear along section L M.

Allowable shear = 50 pounds per square inch.

The average pressure $= \frac{6\frac{1}{3}\ x\ 62.5}{2} = 197.5$ pounds.

Total shear tending to cause shear- considering that the earth back ofthe wall will not support the wall at all thus giving the worst possible condition of design as a factor of safety.

$$\text{Shear} = \frac{6\frac{1}{3} \times 62.5}{2} \times 6\frac{1}{3} = 1255 \text{ pounds.}$$

$$\text{Unit Shear} = \frac{V}{b\ j\ d} = \frac{1255}{12 \times 7/8 \times 7} = 17 \text{ pounds per sq. in.}$$

(d) Summation of moments around L = zero., also considering no fill as supporting the wall.

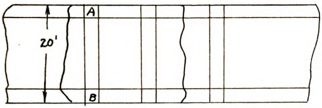
$8 \times 8/12 \times 1 \times 150 \times 2 = 1600$ pound feet.

$$K = \frac{M}{b\ d2} = \frac{1600}{12 \times 7 \times 7} = 2.72.$$ Actually K may be 107.4#

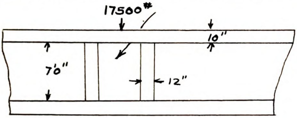
$A_s = p\ b\ d = .0094 \times 12 \times 7 = .7896$ sq. in.

Reinforce with electric welded steel fabric of such size as to care for this design requirement. No. 4.

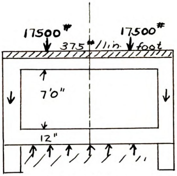
Design of Spillway slab:



Plan of Spillway Slab (Cut-away)



Cross Section



End View

Loads:

Uniform load on pavement slab = 375 x 2 x 10 = 7500#

Concentrated load on slab = 17500 x 2 = 35000#

Load due to weight of wall (AB) 20x7x1x150 = 2100#

Weight of 10" slab = $10 \times 10 \times 150 \times \frac{10}{12}$ = 12500#

Then the load per foot = $\frac{57100}{20}$ = 2850 #/ft.

Moment: = $\frac{1}{8} w l^2$ assuming that the action to be that of a simple beam over the supports to prevent possible cracking of the slab.

$$M = \frac{2850 \times 20 \times 20}{8} = 142500 \text{ \# ft.}$$

$$bd^2 = \frac{M}{K}$$

$$d^2 = \frac{142500 \times 12}{164 \cdot 12} = 870 \text{ sq.in.}$$

d = 29.5 inches. Actually make it 7' o" thick, the height of the water passage which form s the spillway proper.

Reinforce with p = 9 x 29.5 x .0094 = 2.5 sq. in. Use 7/8" round bars @ 6" c-c. in two layers 2" apart.

The design of the Penstock:

Although the size of steel pipe to use for a given discharge varies within wide limits, there generally is one size that will make for the greatest economy of design. The velocity range varies from 8 to 12 feet for heads up to 200 feet. The turbine requirements, indicated by the shape of the load curve, shows a peak load of comparatively short duration. In other words this station is to be used for peak load coverage and is intended to serve as a n interconnected station as has already been explained. This fact influenced the design to a great extent because a higher velocity is permissable and also more economical.

Enger* in an article in the Engineering News Record Volume 70 page 300 derived an equation for the economic diameter of steel pipe lines. This equation follows:

$$d = 9.08 \left\{ \frac{e\ b\ C\ Q^3}{a(R-i)\ t\ c^2\ (1-n)} \right\}^{\frac{1}{6}}$$

in which

d = most economic diameter in inches,

e = overall efficiency of plant to point of sale, estimated to be 75% in this problem,

b = value of lost energy in mills per kilowatt hour at point of sale, estimated by Burd* to be 7 mills.

C = coeff. for use in determining the productive head the value of which is 2.0,

Q = average discharge in cubic feet per second, assumed to be 300.

a = cost of steel in the pipe in dollars per pound, the value of which is estimated by Gillette to be 0.1¢

* Edward M. Burd, in Michigan Engineer, see reference sheet.

R = desired return on money invested (.15%)

i = estimated annual operating, tax, and depreciation charges in per cent of construction cost, expressed as a decimal (.02)

t = thickness of pipe (.375")

c = Chezy coefficient (110)

n = percent overweight due to rivets, overlap, etc., (.2)

Substituting these values in the above equation-

$$d = 9.08 \quad \frac{.75 \times 7 \times 2 \times 300 \times 300 \times 300}{.1 \times .17 \times .375 \times 110 \times 110 \times 1.2}$$

$$= 9.08 \ (3,060,000)^{\frac{1}{6}}$$

$= 109"$ Use a nine foot penstock pipe of 3/8 inch thickness.

The foregoing method, although being of an approximate nature due to the assumptions made in its solution, is considered to be accurate enough for purposes of design to follow.

The following stresses taken from the specifications of the Pacific Coast Electric Association are used in the design of the pipe.

	Ult. Str. #/sq. in.
Tension	55,000 " " .
Shear	44,000 " " .
Bearing	95,000 " " .

The details of the design are given here in accordance with the specifications already mentioned.

The rivet hole diameter shall be one sixteenth of an inch larger than the shank of the cold rivet.

Punching and Reaming: All holes shall be sub-punched and reamed for a butt joint. All holes for lap joint of seven-sixteenths inch or more shallbe sub-punched and reamed. Holes in lap joints pipe of three-eights inch thickness or less may be punched to size.

Deductions for net area: For punched holes, a deduction for hole of three-sixteenths inch greater diameter than the cold rivet shank diameter is made in the computing the net area. For sub-punched and reamed holes, a deduction for hole of one-sixteenth inch greater than the cold rivet shank diameter is made in computing the net area.

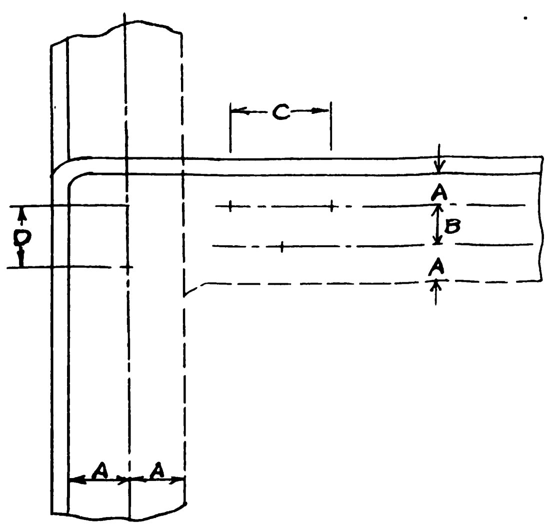
Slope distances: Edge distances are at least one and a half times the diameter of the hole.

Rivet spacing: The distances between the row of rivets is such that the sum of the two net diagonal distances between holes will not be less than 1.25 times the net distance between the holes measured on gage lines.

The maximum spacing of holes along caulked edges is governed by the formula $P = 2\frac{1}{2}\,t - d - 1\frac{1}{2}''$.
in which t = plate thickness; d = diameter of rivet holes; and p = pitch.

All rivets spaces shall be great enough to permit the use of standard rivet dies.

Data for Double Riveted Lap:



t = 3/8 inches.
d = 3/4 " .
A = 1½ " .
B = 1 11/16 " .
C = 3 1/16 " .
D = 2½ inches .
e = 70%

Using the diagram on page 432, Creager and Justin, Hydro-Electric Handbook, for the maximum head on the pipe in feet of water, the following results were obtained.

Using a weight of pipe of 470 pounds per feet and a maximum head of 70 feet, a thickness of pipe of 3/8 inches a double lap joint and two plates, the results for the tension are:

E = efficiency of joint in decimal,

H = loading in feet of water,

d = diameter of conduit in feet,

T = tension in pounds per lineal foot of conduit on each side of conduit,

p = 15000,

A = required grossarea of steel per lineal inch of conduit on each side,

Then:

Tension per lineal inch = 2.6 H d

= 170 x 2.6 x 9

= 3980#.

Tension per lineal foot = 12 x 3980 = 47700#.

To remove the possibility of damage to the pipe line which would be caused by collapse should water be removed from the pipe faster than it could be supplied by the fore-bay, an air inlet valve is placed in the pipe line.

There are two possibilities for the use of the air in-let valve. If it is desired to empty the pipe line for any reason, the air inlet valve is opened to allow air to enter into the pipe as the contents of the pipe are discharged through the turbine.

The second case, one not so important and less likely to occur, is in case of rupture at any point in the line.

The type of valve chosen in this design is manufactured by the Barrett Machine Company of Pittsburgh Pennsylvania. The type is the common gate valve and is a standard 24" valve. If it were not for the excessive height required where the pipe line is at a considerable distance from the water sur-face, a stand pipe would be used in preference to a valve.

This valve is to receive frequent inspection as the pipe line is emptied.

The computations for determining the size of the air inlet are given below.

Let Q = flow of air through the air inlet in cubic feet per second,

c = coefficient of discharge through the air inlet,

F = area of air inlet in square feet,

P = safe difference in pressure between the inside and the outside of the pipe in pounds per square inch,

t = thickness of steel pipe in inches,

d = diameter of pipe in inches,

s = factor of safety against the collapse of the pipe,

Then:

$$F = \frac{Q\ s}{2,460,000\ c} \quad \frac{d}{t} \quad *$$

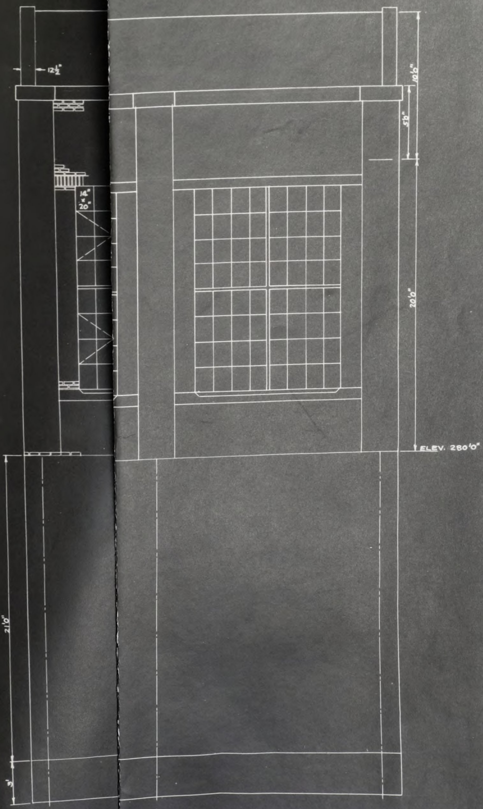
$$F = \frac{500\ 5}{2460000 \times .5} \quad \frac{108}{.375}$$

F = 2.6 feet. Use a 24 inch valve.

*Engineering News Record, Volume 69, 1914. Page 594.

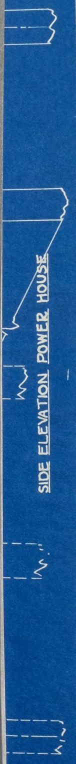
CHAPTER V1

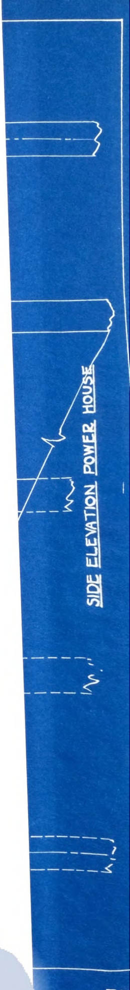
THE DESIGN OF THE POWER HOUSE



PLATE

ELEVATION - POWER HOUSE. Fig. 10





Fig

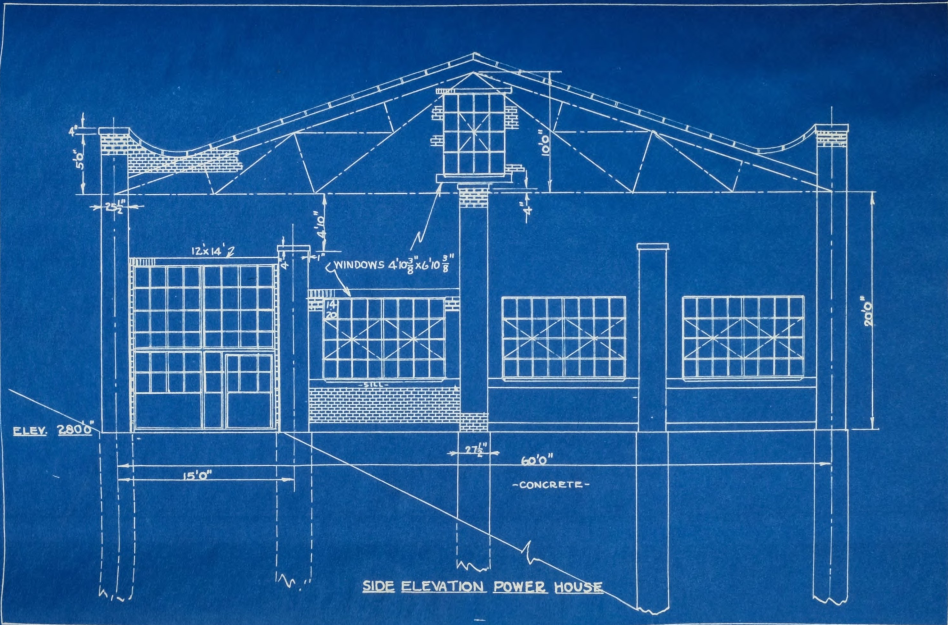

4"
5'0"
25½"
4'10"
12'x14'2
4"
1"
WINDOWS 4'10⅜" x 6'10⅜"
14
20
-SILL-
10'0"
4"
20'0"
ELEV. 2800'0"
27½"
60'0"
15'0"
-CONCRETE-
SIDE ELEVATION POWER HOUSE

Fig. 12

Fig. 13.

The design of a Steel Roof Truss for Power House.

The design of a Fink roof truss for the power house roof, of span 60 feet and rise 10 feet , and distance center to center of trusses 15 feet, is given in detail in the design which follows and in the accompanying drawings E, F, and G.

The roof covering consists of corrugated steel sheets supported directly on the purlins. A lining made of felt two layers thick and two layers of tar paper are placed directly upon wire netting that is stretched over the purlins and is required to prevent condensation under the metal roofing and also to act as an insulator. An allowance of $1\frac{1}{4}$ pounds per square foot is made for this lining. The snow load varies with the geographic location, with the altitude of the structure and also with the slope of the roof. The value chosen in this design is the maximum value given in specification* namely 25 pounds per square foot of horizontal covered surface.

As for the load on the truss caused by the wind, the assumption is made that the maximum value for the velocity of the wind is such as to cause a pressure of 30 pounds per square foot on a vertical surface. The normal wind pressure is then on an inclined surface of 18° 26', 18.37 pounds per square foot.

Design of purlins:

The slope of the upper chord is computed as follows: Length of upper chord = $30^2 - 10^2$ = 31.62 feet. The cosine of the slope angle equals $\frac{30}{31.62}$ = 18° 26'. The panel length from the length of the upper chord is one fourth of 31.62'

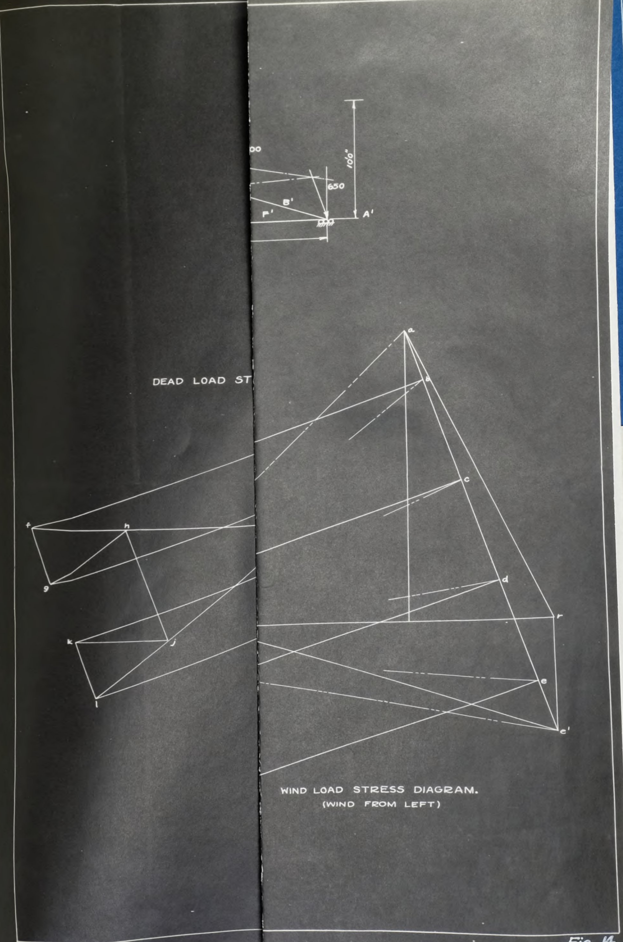
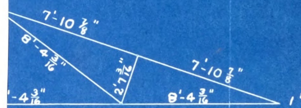


Fig. 14.

PLATE E. OF ROOF TRUSS FOR POWER HOUSE.

Fig. 15



T. STRESS AND FRAMING DIAGRAM.

Fig. 15

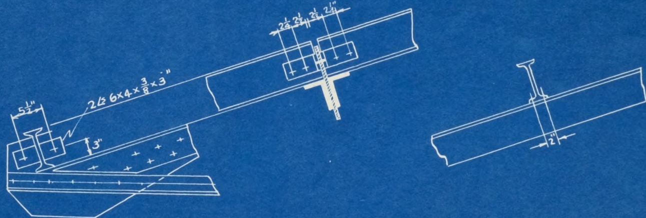


FIGURE 13

Fig. 27

or 7.9 feet. Since the corrugated metal sheeting is placed directly on the purlins, it is necessary to place intermediate purlins other than at the panel points. These are placed at the third points. The gage to use for the metal in these sheets is calculated as follows: $\frac{7.9}{3} = 2.63$ feet. Therefore 22-gage metal corrugated sheeting is allowable since the span allowable for this span is given in the specifications as 4.0'.

Three conditions of loading are investigated in the design of the purlins: (1) dead load and snow load; (2) dead load and wind load; (3) dead load, minimum snow load, and wind load. The computations will be referred to as the number of the condition of loading.

The roof area covered by one purlin is 2.63 x 15 = 39.45 square feet. The horizontal projection of the roof area supported by one purlin is $2.63 \times \cos(\tan^{-1} \frac{1}{3}) = 37.45$ square feet. Assuming a maximum wind velocity of an amount to cause the pressure on a vertical surface of 30 pounds per square foot, and the value of the snow load already referred to as 25 pounds per square foot of horizontal covered surface, the loads on one purlin are as follows:

(1) Roof covering 1.75 x 39.45	= 69.04 #
Roof lining 1.25 x 39.45	= 49.31 #
Snow 25 x 37.45	= 936.25 #
Assumed weight of one purlin	=180.00#
TOTAL	1235 #

(3) Vertical load;

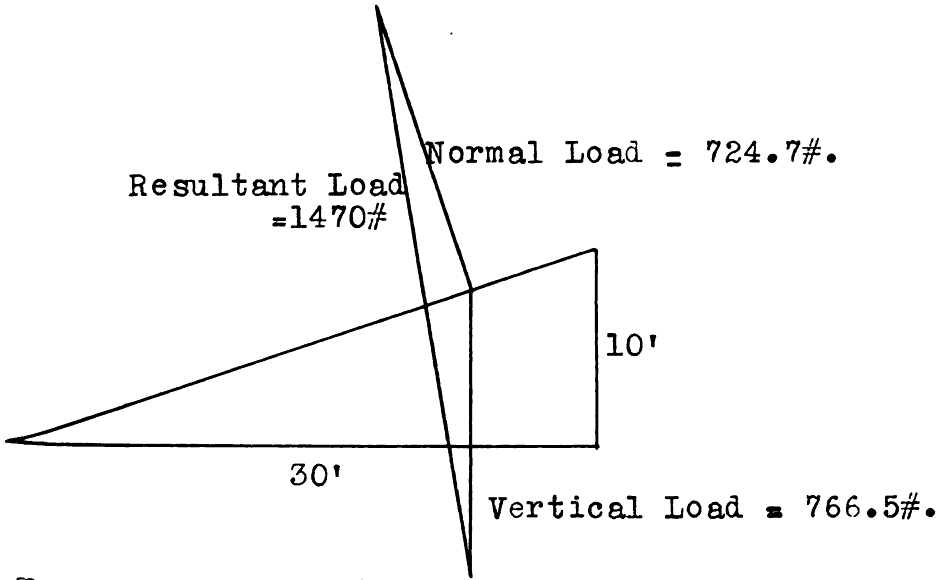
Roof covering	= 69.04 #
Roof lining	= 49.31 #

One half snow	= 468.12 #
Assumed load of one purlin	= 180.00 #
TOTAL	766.50 #

Normal Load:

Wind pressure 18.37 x 39.45 = 724.7 #

The resultant of these two loads for case (3) is determined graphically as shown in the sketch below and is equal to 1470 pounds.



Treating the purlin as a simple beam, supporting a uniform load, the maximum bending moments for the two conditions of loading are:

(1) $M = \frac{1}{8} \times 1245 \times 15 \times 12 = 28{,}012$ pound inches.

(3) $M = \frac{1}{8} \times 1470 \times 15 \times 12 = 33{,}175$ pound inches.

Selection of purlin:

Let M = moment due to the normal load,

M' = moment due to the longitudinal load (hor.)

S = the section modulus about the purlin axis parallel to the roof surface.

Computations for weight of roof truss:

No.	Member	Weight	Length	Total Weight #
8	2 angles 5 x 3 x 7/16"	11.3 #/'	7'10 7/8"	1430.0
4	2 angles 3 x 2½ x 5/16"	5.6 #/'	8' 4 3/16"	374.0
1	2 angles 3 x 2½ x 5/16"	5.6 #/'	26' 8 3/8"	299.0
4	1 angle 2 x 2 x ¼"	3.19#/'	2' 7 3/16"	33.2
4	2 angles 2 x 2 x ¼"	3.19#/'	8' 4 3/16"	213.0
2	2 angles 2½ x 2 x ¼"	3.62#/'	5' 3 9/16"	76.7
4	2 angles 2 x 2 x ¼"	3.19#/'	8' 4 3/16"	213.0

Weight of rivets = 28# (length under heads 2 1/8")	28.0
Weight of heads	9.7

Plates:	Size	
4	6 x ½ x 9¾"	16.6
2	15½ x ½ x 4'1"	108.0
2	9½ x ½ x 7"	9.35
2	11½ x ½ x 8"	13.02
1	36" x 18 x ½"	45.90
2	16 x 8½ x 3/8"	29.0
2	16 x 5/8 x 11 3/8"	11.30
2	10 x 5/8 x 11 3/8"	40.40

Shoe Angles:

2 angles 3 x 3 x 3/8" 10 feet in length	144.0
	3131.87#

Estimated weight as use in computations for size of members

3376.0#

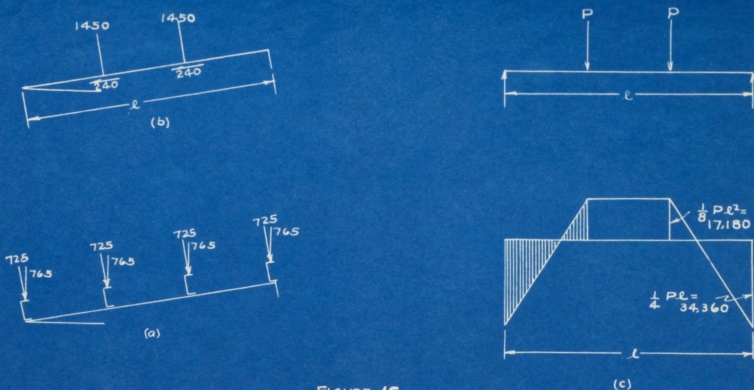


FIGURE 16

Fig. 16

S' = the section modulus of the same around the purlin axis, normal to the roof surface.

Then, $f_s = \frac{M}{S} + \frac{M'}{S'}$

For I-Beam sections, the assumption may be made that S = 7 S'; and for channel sections, S = 10 S'.

Applying the above method to the selection of the purlin in this design,

(1) Normal load = 1245 x .94869 (cos angle) = 1181#

Longitudinal load = 1245 x .3162 (sin.) = 393.7 #

$M = \frac{1181}{8} \times 15 \times 12 = 26572.0\#$

$M = \frac{393.7}{8} \times 15 \times 12 = 8885.0\#$

$S = \frac{(26572 + 7 \times 8885)}{18000} = 4.93 \text{ in.}^3.$

A 5"- 12.25 pound I-Beam is the lightest section modulus that is equal to or greater than that required. For this section, S = 5.4 in.^{3}, and S' = .91 in.^{3}.

(3) Normal load = 724.7 - 766.5 x .94869 = 1451.87 #

Longitudinal load = 766.5 x .3162 = 242.37 #

$M = \frac{1451.97}{8} \times 15 \times 12 = 32667.0\#$

$M' = (\frac{242.37}{8} \times 15 \times 12 = 5453.30\#$

$S = \frac{(32667 + 7 \times 5453.3)}{18000} = 3.93 \text{ in.}^3.$

Therefore the I-beam, 5"-12.25# I-Beam also satisfies this condition of loading.

Stresses in the members of the truss:

Probable weight of truss $w = \frac{p\ l}{150 + 5l + \frac{ps}{3}}$

$= \frac{40 \times 60}{150 + (5 \times 60) + (\frac{40 \times 15}{3})}$

= 3.69 pounds per sq. ft. of horizontal covered surface.

and the total weight of one truss figures 3.69 x 15 x 60 = 3376 pounds. The computed weight of the truss is included in a later page.

The proportion of the total weight which is assumed to be concentrated at each intermediate panel point on the upper chord is $\frac{1}{8}$ x 3376 = 422 pounds. One half of the we ight is assumed to act at the end of each end panel point. Each intermediate panel point supports 7.9 x 15 = 118.5 square feet of roof surface, and 118.5 x .94869 = 112.4 square foot horizontal projection of the roof surface. The dead, snow, and wind panel loads are tabulated on the suceeding pages.

The wind loads caused by the wind coming from the left of the truss are the only ones tabulated since this is the fixed end of the truss and the wind loads caused by the wind coming from the right end are less than the loads caused by the former loading as was proved by computations not included here.

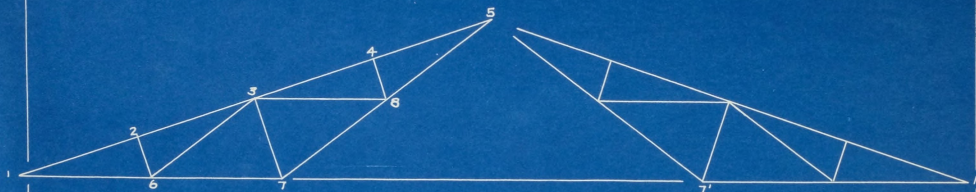
Corrugated metal sheathing 118.5 x 1.75	=	207.4 #
Roof lining 118.5 x 1.25	=	148.12 #
Purlins 3 x 15 x 12.25	=	551.25 #
Truss, per panel	=	422.00 #
Total dead panel load		1328.77 #
Snow panel load = 112.4 x 25		2810.00 #
Wind panel load = 118.5 x 18.37		2176.85 #

The end panels in each case are loaded with one-half the amounts shown above and the wind load at the peak points is one-half of the value computed for the intermediate panels. The loads used in the design in the determination of the stresses are then: 1300#; 2800#; 2200#.

STRESS DIAGRAM

Member	Dead Load	Snow Load	½ Snow Load	Wind Load	Dead - Snow Load	Dead - Wind	Dead ½ Snow Wind
B F	-14480	-31175	-15587	-15600	-45655	-30080	-15667*
C G	-14060	-30270	-15135	-15600	-44330	-29660	-45795
D K	-13640	-29367	-14684	-15600	-43007	-29240	-43924
E L	-13240	-28506	-14253	-15600	-41746	-28840	-43093
F R	13760	29625	14913	17300	43385	31060	45873
H R	11840	25492	12746	13780	37332	25620	38366
M R	8080	17396	8693	6900	25476	14980	23673
FG=KL	-1200	-2584	-1292	-2200	-3784	-3400	-4692
H J	-2400	-5167	-2583	-4400	-7567	-6800	-9383
GH=KJ	1900	4090	2045	3480	5990	5380	7425
J M	3760	8095	4047	6880	11855	10640	14687
L M	5700	12272	6136	10400	17972	16100	22236
AR Vert.	5299	11195	5597	6100	16395	11300	16897
AR Hor.	....	.....	....	2900	.....	2900	2900
AR Reslt.	....	.....	....	6660	.....	11860	17100
A'R	5200	11196	5597	2320	16395	7520	13117

* Indicates maximum stress.



FRAMING PLAN FIG. 4 F.

Fig. 17

Method of Framing:

Each member of the truss, except the secondary atruts F G, and K L, are made of two angles, and the connection at the joints is made by placing gusset plates between the vertical legs of the pairs of angles. Because of the relatively small stresses in the comparatively short members F G, and K L, these members are made of one angle, one leg of which is riveted to one side of the gusset plate at either end. For structures of this type, since the span is less than 75 feet, a size of rivet of 5/8 inches is used. The minimum leg in which this size of rivet can be driven is 2 inches. While in most of the members, rivets are required in one leg only, the smallest angle of standard size that may be used is one therefore, with one leg of 2 inch size. The smallest angle used in this design is 2 x 2 x ¼ inches.

In order to facilitate shipping, the entire truss will be assembled as shown in Drawing . The parts 1-7-5 and 1'-7'-5' may be laid on a flat car on the sides 1-5 and 1'-5' and the piece 7-7' may be shipped loose. With this arrangement, the joints 7, 7', and 5 will have partly field and partly shop driven rivets as is shown on the drawing. The gusset plates at each joint are made ¼ inch, except those at the joints 1, 7, 5, 7' and 1' which are made 3/8 inches in thickness to avoid using large plates to accomodate the required number of rivets. The members and joints in the left half of the truss will be made the same size as the corresponding members and joints in the right half. The upper chord will be made in one piece; the lower chord 1-7 and the diagonal 7-5 are also made continuous across the middle joints of

these members. The additional rigidity and resulting saving in shop work more than compensates for the slight excess of metal that is furnished in the least stressed portions of the truss.

Design of tension members:

The allowable unit stress of the net section is 18000 pounds per square inch. In all members except 6-7, one rivet hole only need be deducted from the gross area of each angle. The effective diameter of hole, for 5/8 inch rivets = $\frac{5}{8} - \frac{1}{8}$ equals $\frac{3}{4}$ inches. The computations made for each member are as follows:

Member 1-7

The maximum stress = 45,873 #.

The net area required = $\frac{45873}{18000}$ = 2.55 square inches.

Two angles 3 x $2\frac{1}{2}$ x 5/16 inch gross area = 3.24 sq. in.

The net area furnished = 3.24 - 2 x $\frac{3}{4}$ x 5/16 = 2.77 sq. in.

The portion 6-7 of the continuous member 1-7 has a less stress than the above, but as will be noticed later, at joint 7, rivets are placed in both legs of the angle comprising this member, and the net area furnished at this point is less than that indicated above, Assuming that the rivets in both legs of the angle occur in the same cross section, a total of two holes must be deducted from the angle. And since there are two angles, the net area furnished is equal to:

3.24 -4 x $\frac{3}{4}$ x 5/16 = 2.3 sq. in. The net area required = $\frac{38366}{18000}$ = 2.13 sq. in. wjich is less than that furnished. Place the 3 inch leg in the vertical position to give greater flexural strength.

Member 7-7'

Member 7-7' cont.

In order to facilitate the riveting at the joint 7, this member is made of the same size angles as used in the member 1-7, two angles $3 \times 2\frac{1}{2} \times 5$ inches.

The maximum stress = 23,673 #.

The net area required = $\frac{23673}{18000}$ = 1.32 sq. in.

The net area furnished, allowing for 2 rivet holes in each angle is 2.3 square inch as computed above. While this is much in excess of the area required, the member 7-7' is but a small proportion of the entire truss and the excess weight adds but little to the total cost of the truss.

Members 3-6 and 3-8:

The maximum stress = 7425 #.

The net area required = $\frac{7425}{18000}$ = .41 square inches.

Two angles $2 \times 2 \times \frac{1}{4}$ inches gross area = 1.88 square in.

The net area = $1.88 - 2 \times \frac{3}{4} \times \frac{1}{4}$ = 1.5 square inches.

As stated above, these are the smallest size angles in which the required number of rivets can be driven.

Member 5-7:

The maximum stress = 22,236 #.

The net area required = $\frac{22236}{18000}$ = 1.24 square inches.

Two angles $2 \times 2 \times \frac{1}{4}$ inches as above furnishthe net area of 1.5 square inches.

Design of Compression Members:

The allowable unit stress on the cross section of members in compression is

$$s_c = \frac{18000}{1 + \frac{l^2}{18000\, r^2}} \quad \text{(from specifications)}$$

The maximum value of the above is given at 15000 #/sq.in.

The specification also limits the ratio of length to least radius of gyration to a maximum of 120. The computations for the members of the truss follow.

Members 2-6 and 4-8:

These secondary members of the truss are to consist of a single angle. The maximum stress is 4692 #. Assuming the allowable units stress to be 10000# per square inch, the gross area required = $\frac{4692}{10000}$ = .47 square inches. The length of the member (see framing diagram) is 31 inches. In order that the ration of l/r, the slenderness ratio, shall not exceed 120 the selected angle must furnish a least radius of gyration being .26 inches, that is $\frac{31}{120}$. The smallest angle which satisfies this last requirement is a $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ the least radius of gyration being .29 inches. But this angle is smaller than the minimum allowed. The selected angle is a $2 \times 2 \times \frac{1}{4}$ the least radius of gyration being .39 inches, gross area of .94 inches. The true allowable unit stress is then:

$$\frac{18000}{1 + \frac{31 \times 31}{18000 \times .39^2}} = 13000 \text{ \#/ square inch.}$$

and the gross area furnished is ,94 inches and the area required is $\frac{4692}{13000}$ = .35 square inches.

Member 3-7:

Maximum stress = 9383 #. The length 63.6 inches.

The minimum allowable value of "r" = $\frac{63.6}{120}$ = .527

Assume the allowable unit stress as 10000 pounds per square inch.

The gross area required = $\frac{9383}{10000}$ = .94 square inches.

Two angles $2\frac{1}{2} \times 2 \times \frac{1}{4}$ inches with the long legs sep-

arated by a $\frac{1}{4}$" gusset plate, furnish the least radius of gyration equalling .89 inches. and a gross area of 2.12 squabe inches. The true allowable unit stress is then:

$$\frac{18000}{1 + \frac{63.6 \times 63.6}{18000 \times .89 \times ,89}} = 14000 \text{ pounds per square inch.}$$

and the true gross area required $= \frac{9383}{14000} = .67$ sq. in.

A smaller angle can not be used because of the requirement of the smallest radius of gyration, and the excess area of steel which id furnished is unavoidable.

Design of the Upper Chord:

Because of the arrangement of the purlins, some of which rest directly on the upper chord at other then panel points, this member if the truss is subjected to bending stresses in addition to the direct compression which is caused by its position in the truss.

The equation for the combined required gross area

$$A = \frac{N}{f_1} + \frac{Me}{f_2 r^2}$$

Since the upper chord is to be made in one length, the conditions of loading and support that justify the assumption of fixity at the ends of the individual panel points. While this assumption reduces the amount of maximum bending moment as compared with a condition of simple support, yet it necessitates investigation for bending both at the center and at the end of each panel. This gives the result for positive bending moment in the first case and for negative bending moment in the second case, Since the direct compression in the end panel 1-2 is greater than in any other panel of the truss, the entire chord section will be fixed by the require-

ments of this one member.

The stress sheet shows that the combination which is the required loading is a combination of wind, dead, and minimum snow load.

The loads on each purlin are shown in the figure . The total normal concentration at each third point of the upper chord member 1-2 = 725 + 765 x .9492 = 1450 #., and the total longitudinal concentration at each of these points = 765 x .3162 = 240 pounds.

Specifications allow for a reduction in bending moment computed as for simple beams in cases where a member is continuous over panel points. Therefore the maximum bending moment in the continuous beam may be taken as $\frac{3}{4}$ of the maximum in the simply supported beam. The maximum bending moment in the continuous beam occurs at the supports, and for symmetrical loads, the moment at the canter may be taken as one-half of the end moment. With the present arrangement of loads the moment at the support equals:

$.75 \times 1450 \times \frac{7.9}{3} \times 12 = 34{,}360$ pound inches; that at the center of the panel equals

$34{,}360 \times .5 = 17{,}180$ pound inches.

Since the member 1-2 is essentially a compression member, the maximum combined stress occurs in the extreme compression fiber and the value of "e" must be selected carefully. At the center of each panel, the value of "e" is thus measured from the gravity axis to the top of the member, and at the support it is measured to the bottom of the member.

The size of the member must be fixed before these values are known. Assume two angles $4\frac{1}{2}$" x 3" x 5/16", the long

legs separated by a $\frac{1}{4}$ inch gusset plate and the short legs outstanding to give greater resistance to the direct stress and also to that caused by the bending. The radius of gyration = 1.44 inches about axis 1-1; and 1.22 about axis 2-2. The value of "e" at the center = 1.47 and at the end equals 3.03. At the center the total direct compression is equal to the maximum compressive stress in the member = 45667 #. The total direct compression at the end of the member = 45667 - 240 (the longitudinal compression component). equals 45900 pounds. Since the member is assumed to be rigidly fixed at the end, no column action need be considered there, and the allowable compressive unit stress is 18000 pounds per square inch. For the section as assumed above, the values of l and r are 7.9 x 12 = 94.8 inches and 1.44 inches respectively.

$$\text{Stress} = \frac{18000}{1 + \frac{94.8 \times 94.8}{18000 \times 1.44 \times 1.44}} = 14500 \text{ pounds.}$$

The value of "r" about the axis 2-2 is less than it is about axis 1-1/ However the purlins may be assumed to furnish lateral support about the axis 2-2, and for the the bending about the axis, the unsupported length may be taken as $\frac{1}{3}$ x 94.8 = 31.6 inches. When substituted in the column formula above, these values result in a higher unit stress than when r_{1-1} and a length of 94,8 inches are used.

The allowable unit fiber stress in bending is 18000 pounds per square inch, and the radius of gyration about the axis of bending = 1.44 inches. The above values are assembled below and the areas required at the end and at the center of

the member are determined as shown.

Investigation at the center:

$N = 45667$ #.

$f_1 = 14500$ pounds per square inch.

$M = 17180$ inch pounds.

$e = 1.47$ inches.

$f_2 = 18000$ pounds per square inch.

$r = 1.44$ inches.

$$A = \frac{45667}{14500} + \frac{17180 \times 1.47}{18000 \times 1.44 \times 1.44} = 3.149 - .67 = 3.8 \text{ sq. in.}$$

Investigation at the end:

$N = 45667$ pounds.

$f_1 = 18000$ pounds per square inch.

$M = 34360$ pound inches.

$e = 3.03$ inches.

$f_2 = 18000$ pounds per squabe inch.

$r = 1.44$ inches.

$$A = \frac{45667}{18000} + \frac{34360 \times 3.03}{18000 \times 1.44 \times 1.44} = 2.54 - 2.79 = 5.33 \text{ sq. in.}$$

The area furnished = 2 x 2.25 = 4.50 inches which is too low.

The next size angle assumed is 2 angles 5 x 3 x 7/16 " the long legs separated by a ¼ inch gusset plate and the short legs outstanding as before. The radius of gyration about the axis 2-2 is 1.19 inches. The value of E at the center is 1.73 inches and at the end is 3.27 inches and the radius of gyration about the axis 1-1 is 1.19 inches. See fig.G

$$\text{Stress} = \frac{18000}{1 + \frac{94.8 \times 94.8}{18000 \times 1.6 \times 1.6}} = 15000 \text{ pounds per sq. in.}$$

Investigation at the Center:

$N = 45667$ pounds.

$f_1 = 15000$ pounds per square inch.

$M = 17180$ inch pounds.

$e = 1.73$ inches.

$f_2 = 18000$ pounds per square inch.

$r = 1.60$ inches.

$$A = \frac{45667}{18000} + \frac{17180 \times 1.73}{18000 \times 1.6 \times 1.6} = 3.68 \text{ square inches.}$$

Investigation at the end:

$N = 45667$ pounds.

$f_1 = f_2 = 18000$ pounds per square inch.

$M = 34360$ inch pounds.

$e = 3.27$ inches.

$r = 1.6$ inches.

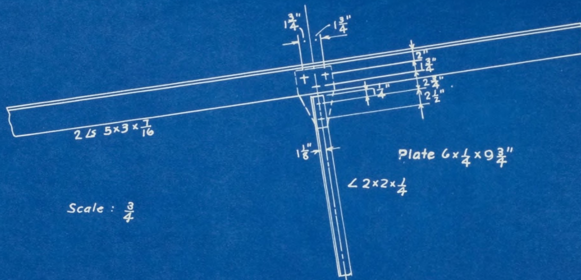
$$A = \frac{45667}{18000} + \frac{34360 \times 3.27}{18000 \times 1.6 \times 1.6} = 2.54 - 2.44 = 5.00 \text{ sq. in.}$$

The area furnished = 2 x 3.31 = 6.62 square inches.

While a comparatively large excess is provided over that required, these angles are used because they have a smaller section than any other satisfactory pair.

Design of Joints:

Connection between the individual joints of the truss is made by the use of gusset plates which are placed between the angles composing the members. Sufficient rivets are required at the end of each member to transfer the stress to the plate. The number and shape of each plate are determined by the number of rivets required in the several members composing the connection to the plate. Where relatively small stresses are involved, $\frac{1}{4}$ inch plates are used, but in cases



JOINTS 2 AND 4
FIG. 6

Fig. 18

where the stresses are relatively large the size is increased to 3/8 inch. All of the shop rivets are power driven while the field rivets are handpdriven.

<u>Joints 2 and 4</u> :

The maximum stress = 4690 pounds.

The allowable unit bearing stress for shop rivets, power driven, and in single shear = 24000 pounds per square inch, and the bearing value of a 5/8 inch rivet on a $\frac{1}{4}$ inch plate = 5/8 x 1/4 x 24000 = 3750 pounds. The rivets through members 2-6 and 4-8 are in single shear, and with an allowable unit shearing stress of 13500 pounds, the single shear value of one rivet is .3068 x 13500 = 4140 pounds. This value, being greater than that d termined by bearing will govern. Thus, at the end of the members 2-6 and 4-8 , $\frac{4690}{4140}$ = 1 rivet. Use two rivets.

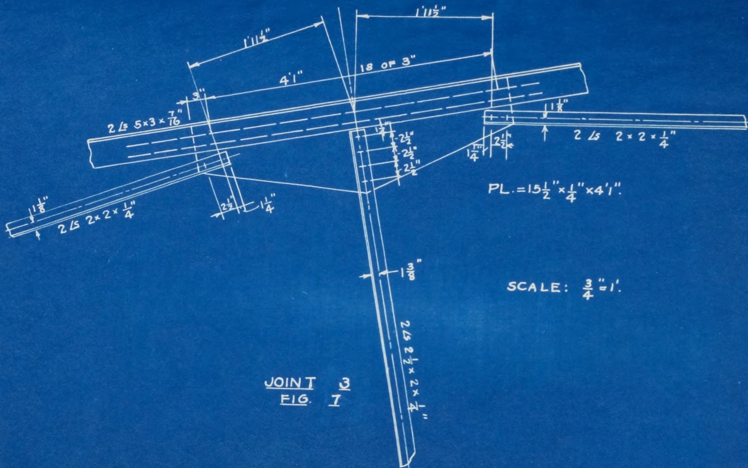
Since the upper chord is pushed against the gusset plate with a force equal to 6300 pounds, there must be sufficient rivets in the chord member to withstand the pressure. The rivets are in double shear and bearing on a $\frac{1}{4}$ inch plate. The latter condition governs, the allowable value of one rivet is 4690 pounds and $\frac{6300}{4690}$ = 2 rivets are required. In the detail 3 rivets are used so as to place the end rivets in the back gage lines of the angles and thus secure a more rigid and symmetrical arrangement. The size of the gusset plate is shown on the detail as is the spacing of the rivets.

<u>Joint 3:</u>

Members 3-6 and 3-8. Maximum stress = 7425 pounds.

The bearing value of one rivet on a $\frac{1}{4}$ inch plate = 4690 #.

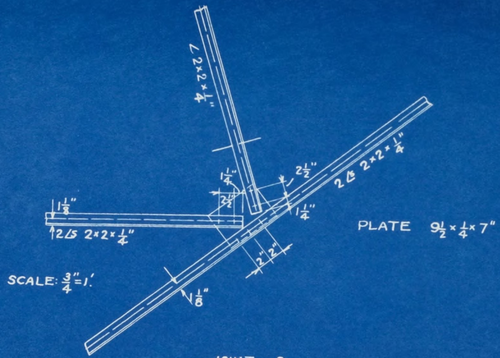
The number of rivets required = $\frac{7425}{4690}$ = 2 rivets.



PL. = 15½"×¼"×4'1".

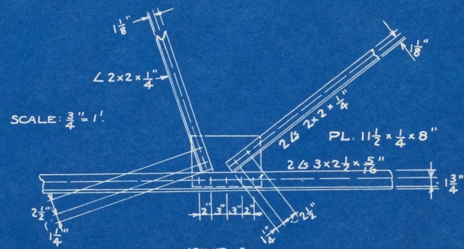
SCALE: ¾" = 1'.

JOINT 3
FIG. 7



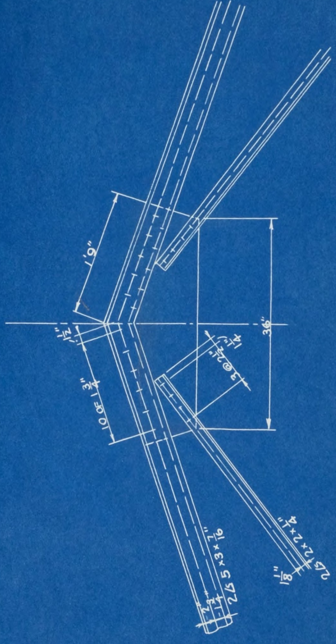
JOINT 8
FIG. 9

Fig. 20



JOINT 6
FIGURE 8

Fig. 21



JOINT 5
FIG. 10

Member 3-7. Maximum stress = 9380 pounds.

The bearing value of one rivet on a $\frac{1}{4}$ inch plate = 4690 #/

The number of rivets required = $\frac{9380}{4690}$ = 2 rivets. Use 3

In the upper chord 2 rivets are required as in Joint 2 but 9 are used in order that extreme rivets be placed in the gage line nearest the plate.

Joint 6:

Member 2-6. 2 rivets are required as determined in the computations for Joint 2.

Member 3-6, 2 rivets are required as determined in the computations for Joint 3.

Lower Chord 1-7. Since this member is continuous across the Joint 6, the difference in stress between the parts on either side of the joint only need be considered in determining the number of rivets through the chord and the gusset plate, namely, 45870 - 38370 = 7500 pounds.

The bearing value of one rivet on a $\frac{1}{4}$ inch plate = 4690 #.

The number of rivets required = $\frac{7500}{4690}$ = 2 rivets. Use 3 in order to fill up the plate.

Joint 8:

This joint is made the same as Joint 6

Joint 5:

The rivets in the members of the left portion of the truss at Joint 5 are shop rivets, those in the right half field rivets. To keep symmetry, the same number are placed in both sides, hand-driving governing.

Member 4-5:

The maximum stress = 43000 pounds.

The bearing value of a 5/8 inch rivet on a 3/8 inch plate

is equal to 4690 pounds.

The number of rivets required $= \frac{43000}{4690} = 9$ rivets.

Make the number 11 to care for bending stress caused by loads on the purlins at the third points of the member 4-5.

Member 5-8:

The maximum stress = 22200 pounds.

The bearing value of one rivet as before = 4690 pounds.

The number of rivets required $= \frac{22200}{4690} = 5$ rivets.

Joint 7.

Member 3-7:

The maximum stress = 9380 pounds.

The bearing value of one rivet on a 3/8 inch plate = 7030#/

The number required $= \frac{9380}{7030} = 2$ rivets.

Member 7-8:

The meximum stress = 14700 pounds.

The bearing value of one rivet as before = 7030 pounds.

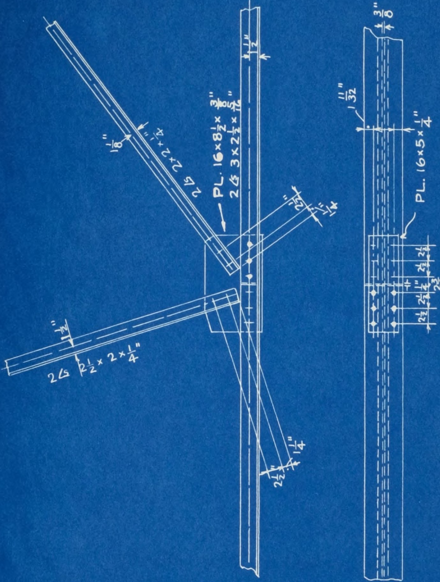
The number required = 2 rivets.

Member 6-7:

In order to reduce the size of the gusset plate at the Joint 7 part of the stresses in the lower chord members will be transferred across the joint through a ¼ inch splice plate under the lower chord. The rivets in the splice plate through member 7-7' are shop rivets and those through member 6-7, are field rivets.

Six field rivets will be placed in the splice plate on the left side of the joint, and 4 shop rivets on the right side. The strength of the plate = 4 x 3750 = 15000 pounds.

The amount of stress still to be provided for from the member 6-7 is 38370 - 15000 = 23000 pounds. The rivets



JOINT 7
FIG. 11.

Fig. 24.

through 6-7 and the vertical gusset plate are shop rivets, which bear on a 3/8 inch plate = 7030 pounds.

Therefore the number required = $\frac{23000}{7030}$ = 4 rivets.

Member 7-7':

The amount of stress which must be transmitted from member 7-7' by the rivets in the gusset plate = 23760-15000. The value of one rivet (hand-driven in bearing on 3/8 inch plate) is equal to 4690 pounds.

The number required then = $\frac{8670}{4690}$ = 2 rivets.

Design of end joint and bearing:

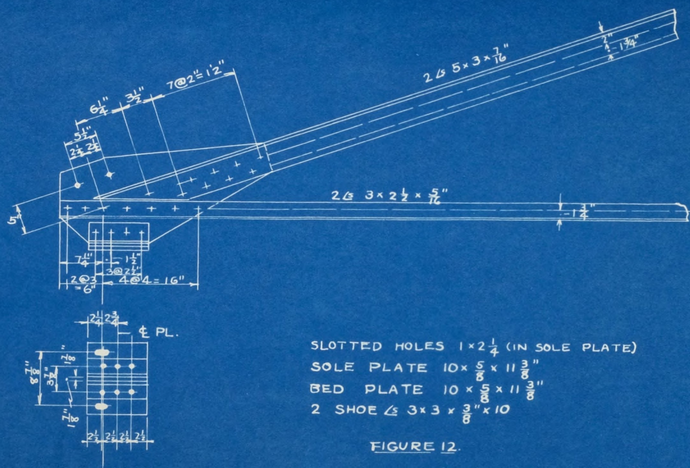
The 3/8 inch plate at the end joint is extended below the lower chord member to permit riveting of a pair of shoe angles to it, the purpose of which being to transfer the end reaction to the gusset plate. A sole plate is riveted to the bottom of the outstanding legs of the shoe angles to stiffen this portion of the angles. The sole plate is allowed to rest on a bed plate which is anchored to the masonry.

The number of rivets required to take care of the compressive stress in member 1-2 = $\frac{45670}{7030}$ = 7. Make the number 9 to care for bending stress cause by the loads on the intermediate purlins.

The number of rivets required in the lower chord member equals $\frac{45870}{7030}$ = 7 rivets. This is member 1-6.

A total of $\frac{17100}{7030}$ = 3 rivets is required between the shoe angles and the gusset plate in order to transfer the resultant end reaction to the plate. The shoe angles will be made 3 x 3 x 3/8 inch by 10 inch.

The area of the bed plate is determined by the maximum



SLOTTED HOLES 1 × 2 1/4 (IN SOLE PLATE)

SOLE PLATE 10 × 5/8 × 11 3/8"

BED PLATE 10 × 5/8 × 11 3/8"

2 SHOE ∠s 3 × 3 × 3/8" × 10

FIGURE 12.

vertical component of the end reaction. Assuming an allowable pressure on the masonry wall of 300 pounds per square inch, the area required $= \frac{16900}{300} = 56.3$ square inches. The width of the plate parallel to the span of the truss is made equal to the length of the shoe angle, 10 inches. The lengt h of the plate parallel to the wall must be made sufficient to provide for $\frac{3}{4}$ inch anchor bolts outside the shoe angles. A length of 6 3/8 - 2 x $2\frac{1}{2}$ = 11 3/8 inches. The bearing thus provided = 10 x 11 3/8 = 113.7 square inches, which is some greater than is necessary.

The thickness of the sole plate a nd of the bed plate is determined by treating the portion projecting beyond the shoe angle as a simple cantilever with an upward load equal to the maximum allowable pressure on the wall, 300 pounds per square inch. For a 1 inch strip of plate $= \frac{1}{6} \times 18000 \times 1 \times t^2$ equals $3000\ t^2$. Solving this equation for t the value is .56 inches. Use a 5/8 inch plate.

For 3/4 inch anchor bolts passing through the slotted holes in the sole plate-to allow for the movement of the truss dur to temperature change and deflection, an allowance of 1 inch per 100 feet of span is ample allowance to make for this. The length of the slotted hole is made equal to the diameter of the anchoe bolt plus twice the expansion. Or L = 2 x .6 plus $\frac{3}{4}$ inches = 1.95 inches. Use 2 inches. The diameter of this hole is 3/4 inch plus 1/4 inch or one inch.

Ordinarily the center of the sole plate would be placed on a vertical line through the intersection of the gage lines of members 1-2 and 1-6. Since, however, a large bending moment is caused by the intermediate purlins in panel 1-2, the

shoe will be placed to the right a distance equal to the moment at the end of the member 1-2 divided by the maximum vertical component of the reaction, $\frac{34360}{16900} = 2$ inches.

Miscellaneous Details:

Each intermediate purlin is riveted to the upper chord as shown in the figure. The end purlin is fastened to an extension of the gusset plate at the end joint by means of a standard connection, as shown in the detail drawing. On account of the arrangement of the truss members at the end joint there is not enough strength in the upper chord to properly support the purlin at this point. Two angles 6 x 4 x 3/8 in. are used for the connecting angles, the length of which are 3 inches. The 6 inch leg is placed along the web of the I-beam and the 4 inch leg along the gusset plate.

All the members which are composed of two angles are riveted at frequebt intervals in order to distribute the stress equally between the two members. In the present case since the angles are separated by gusset plates, a washer is placed between them at each rivet in order to maintain equal distance between the angles. The rivets which are used for this purpose are of the same diameter as the rivets used at the joints. These rivets are shown on the drawing of the assembled roof truss which accompanies this explanation.

Design of Power House Floors:

Section A. Operating floor (see figure 31) Floor Plan.

The design of the slab shown in Figure 32 is as follows:

Loads carried by the slab:

Live load due to people -------	100 lbs. per sq. ft.
Dead load due to concrete -----	50 lbs. per sq. ft.
Additional load as factor of safety against accidental loads ----	50 lbs. per sq. ft.
TOTAL	200 lbs. per sq. ft.

Take a section along A B C D (section 1-1) on Figure 31, and using a 2000 lb. concrete for which $f_c = 800$ pounds per square inch; f_s equals 16000 pounds per square inch; $n = 15$; and making use of the diagram on Page 340, Hool's "Reinforced Concrete Design ", $K = 147$; $p = .0108$,

Then

$$M = K\ b\ d^2$$

$$M = \frac{W\ l^2}{10} = K\ b\ d^2$$

$$\frac{200 \times 10 \times 10 \times 12}{10} = 147 \times 12 \times d^2$$

$$d^2 = 13.69 \text{ square inches}$$

$$d = 3.7 \text{ inches. Use a 6 inch slab.}$$

Sincethe minimum thickness for a slab in such a location and supporting such indeterminate loads, is around one half a foot, it seems desirable to take this value, and the factor of safety added is assumed to be welcome since there is always a possibility that construction loads may accidentally be placed on the slab over an unsupported region. This is of especial significance in the design of the operating floor.

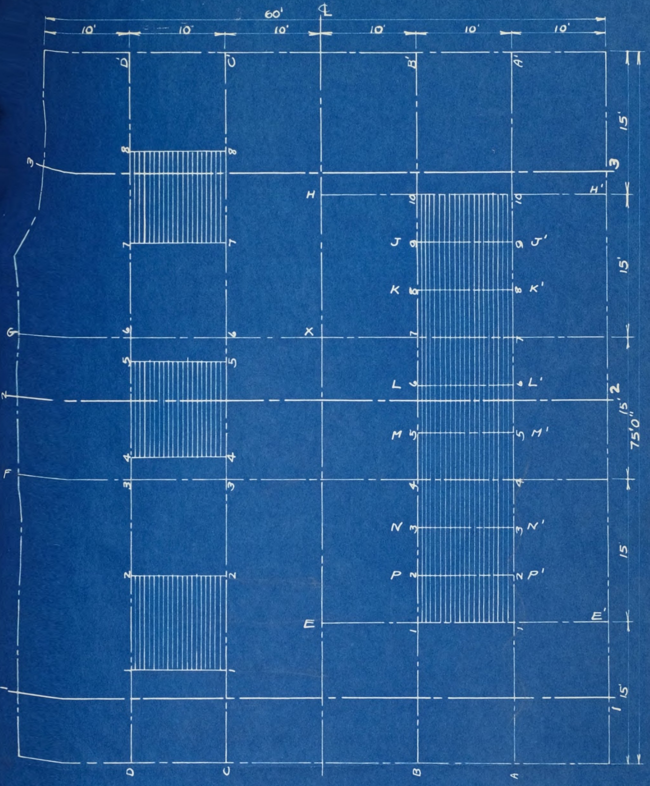


Fig. 31 - FLOOR PLAN OF OPERATING FLOOR

OF

POWER HOUSE

$A_s = p\ b\ d = .0108 \times 3.7 \times 12 = .48$ square inches required. For this steel requirement, ½" square steel bars @ 6" space, are used. This steel furnishes an area of .5 sq. inches which is sufficient. Reinforcement is also placed in the opposite direction to the main reinforcement as shown in the detail of the slab, to take careof temperature stresses and also to balance the reinforcement in the slab proper.

Investigation for shear:

$$v = \frac{V}{b\ j\ d} = \frac{200 \times 5}{12 \times .857 \times 3.7} = 26.3 \text{ lbs. per sq. in.}$$

There fore no diagonal reinforcement is necessary.

Investigation for bond:

The maximum bond stress occurs at the supports in the reinforcement for negative moment. At the supports the unit bond stress is

$$u = \frac{V}{\Sigma 0\ j\ d} = \frac{200 \times 5}{4 \times .857 \times 3.7} = 79 \text{ lbs. per sq. in.}$$

Design of slab at Section 2-2 (see figure 35).

Using a 2000 pound concrete; $f_c = 800$; $f_s = 16000$; $n = 15$; $K = 147$; $p = .0108$ as before.

Then,

$$M = \frac{W\ l^2}{10} = K\ b\ d^2$$

$$\frac{200 \times 10 \times 10 \times 12}{10} = 147 \times 12 \times d^2$$

$d = 3.7$ inches. Use a 6 inch slab as in the previous design with identical reinforcement.

The design of the slab at the sections (a) and (c) as well as at section (b) is the same as the design of the slab at section 1-1. A six inch slab is used throughout the power house operating floor and the details of the reinforcement will be seen by an examination of the detail drawings which have been lettered to correspond with the description here given.

Design of Cross Beams. (A -1) (1-4) etc.

Loads on A-1 $= \frac{10 \times 150}{2} =$ 750 # due to concrete slab.

1000 # due to live load of people.

Total 1750# per foot of beam.

Loads on 1-4 $= \frac{5 \times 150}{2} =$ 375 # due to concrete slab.

500 # due to live load of people.

Total 875 # uniform load per ft. of beam.

Concentrated load on 1-4, = 9.43 x 25 = 235.75 # at third points.

Additional weight of beam = 61.75 # per foot of beam.

Also add 2500# load due to live load of people.

Total 2800 # concentrated at the third points.

Insert: Design of I-beam to support grating.

Load on I = 50 x 9.43 = 471.5 # due to grating.

50 x 100 =5000.0 # due to people.

Total 5471.5 # per foot.

Moment $= \frac{W\,l^2}{8} = \frac{547.5 \times 10 \times 10 \times 12}{8} = 82{,}072$ #".

From diagram use 5" I-beam , weight 12.25 # per foot.

The weight on this I-beam $= \frac{12.25 \times 10}{2} = 61.75$ #.

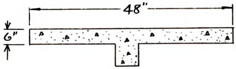
<u>Design</u> <u>of</u> <u>beams</u>. (con't.)

Referring to figure 35 in this set of detail drawings, the following design is of the beams numbered A-1, 1-4.

The bending moment at the center of the fifteen foot span is, as in preceding design considered to be $M = \frac{1}{10} W l^2$.

$$M = \frac{1}{10} \times 1750 \times 15 \times 15 \times 12 = 473{,}000 \text{ pound inches.}$$

Considering an end view of the slab under consideration as given in the sketch below, and using the following values $f_c = 800$; $f_s = 16000$; $n = 15$; $j = .857$; $k = .429$,



$$M_c = f_c(1 - \frac{t}{2kd})\ b.t.j.d.$$

$$M_c = 800\ (1 - \frac{6}{2 \times .429d})\ 48 \times 6 \times .857d = 473{,}000$$

from which

$d = 9.4$ inches.

$$M_s = f_s\ A_s\ j\ d.$$

$$473{,}000 = 16000 \times .0108 \times 48 \times d \times 857\ d$$

from which

$d^2 = 66.4"$

$d = 8.15$ inches.

A check will be made of these values by the use of the diagram on page 321 of Hools "Reinforced Concrete Construction" Vol. 1.

$p = .0108$; $n = 15$; $f_s = 16000$#/ sq. inch. ; $f_c = 800$#/ sq. inch; $\frac{t}{d} = .43$; $k = .43$; $j = .857$; maximum fiber stress in steel corresponding to f_c equal to 800 #/ sq. inch = 15900; maximum fiber stress in concrete corresponding to an $f_s = 16000$#/sq. inch is equal to 805.

Then:

$$\frac{M\ r}{b\ d2} = 147.5$$

$$M = \frac{473,000}{48\ d2} = 147.5$$

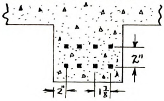
from which

d = 8.15 inches. Add 1.85 inches to make the total depth equal to <u>10 inches</u>. The protective coating makes the depth as shown on the detail drawing of 12 inches.

$A_s = p\ b\ d.$

$$.0108 = \frac{A_s}{48 \times 10} = 5.184 \text{ inches.}$$

For this steel requirement use 8 seven-eighths inch bars at $1\frac{7}{8}$" center to center in two rows as shown in the reinforcement sketch.



<u>Design of beam number 1-4.</u>

Consider the moment in the center of the slab as shown on the drawing of the location of this beam (Fig.5) to be as before; $M = \frac{W\ l^2}{10}$

M = 875 x 15 x 15 x 12 = 236,250 inch pounds.

$b' = 18"$; $f_s = 16000\#$ / sq. in.; $f_c = 800\ \#$ / sq. inch; n = 15; p = .0107; k = .429; j = .857

$$M_c = 236,000 = 800\ (1 - \frac{6}{2 \times .429d})\ 18 \times 6 \times .857d$$

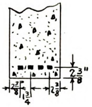
from which

d = 11 inches. Make 12 inches to correspond with the adjacent slab just designed.

$$p = .0107 = \frac{A_s}{18 \times 11}$$

$A_s = 2.118$ sq. inch.

To care for this steel requirement, reinforce with four three-quarter inch bars in a single row spaced one inch between surfaces.(see diagram below)



<u>Design of beam numbered 8-C' (see fig. 1)</u>

The load on beam 8-C' is composed of live and dead loads. The live load is composed of 15 x 10 x 100 = 15000# due to the weight probable by the load of 100# per sq. foot due to people.

The dead load due to concrete is $\left[\frac{(15x5) + (5x10)}{2}\right]$ x 150 pounds equals 9375# due to the weight of the concrete.

The weight of thegrating = 25 x 9.43 = 237.75 #.

From the above computations thetotal load on the slab 8-C' = 24610.75# and this is equal to a load per foot of 1640#.

$M = \frac{1640 \times 15 \times 15 \times 12}{10}$ = 442, 800 pound inches.

For n = 15; f_s = 16000; f_c = 800; p = .0108; j = .857 and k = .43, and a value for $\frac{M}{bd^2}$ = 147.5 from thetable on page 324 of Hool Volume 1 already referred to in preceding design,

$d^2 = \frac{442,800}{7100}$ = 62.2"

d = 7.8 inches. Make the beam 12 inches.

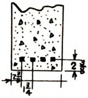
A_s = p b d

= .0108 x 48 x 8

= 4.1472 square inches.

For this steel requirement use 8 three-quarter inch bars

spaced at interval of one inch edge distance as shown in the following sketch of the location of reinforcement.



Design of beam numbered 3-6 (see Fig. 1)

The loads on the beam under consideration are composed of the following:

Load due to concrete $= \frac{15 \times 5 \times 150}{2} = 5625$ #.

Also $\frac{150 \times 5 \times 5}{2} = 1875$ #.

Load due to grating $= 50 \times 9.43 = 471.5$ #.

Load due to live load $= 150 \times 100 = 1500.0$ #.

22971.5 #

This load is equivalent to a load of 1530 # per foot of beam.

$$M = \frac{1530 \times 15 \times 15 \times 12}{10} = 413,000 \text{ \# inches.}$$

$$\frac{M_r}{bd^2} = 147.5$$

From which

d = 7.6 " = 8 inches. Use the same section as in beam 8-C'.

Design of Cross Beams on Operating Floor (see Figure 33)

Design of beam (G—X)

$$M = 12(3075 \times 15 - 80 \times 15 \times \frac{15}{2} - 1875 \times 5) = 333{,}000\# \text{ in.}$$

$$bd^2 = \frac{M}{pf_s j}$$

$$= \frac{333{,}000}{.0107 \times 16000 \times .857}$$

$$= 2260 \text{ square inches.}$$

Assume b = 12", d^2 = 188 from which d = 13.7" = 14" Add 2" for protective coating making thetotal depth = 16inches.

A_s = 12 x 14 x .0107 = 1.8".

For the above steel requirement us e 4 three-quarter inch bars placed at $1\frac{1}{2}$" edge distance. A = 4 x .441 = 1.764 square inches.

The bond for one bar at the left end ofthe beam =

$$u = \frac{V}{\Sigma o\ j\ d} = \frac{3075}{2.356 \times .857 \times 14} = 108.5\ \# \quad \text{O. K.}$$

For plain bars thenumber that must be extended straight to the left end of the beam is

$\frac{108.5}{80}$ = 1.35 = 2 bars. Bend up two bars as a result of the above computations.

$$v = \frac{V}{bjd} = \frac{3075}{12 \times 14 \times .857} = 21.4\ \# \text{ per squareinch.} \quad \text{O. K.}$$

Investigation for bond and shear.

$$v_{A-A} = \frac{V}{bjd} = \frac{1750 \times 7.5}{.857 \times 12 \times 10 \times 12} = 10.65\ \#/\text{sq. in.}$$

$$v_{B-B} = \frac{V}{bjd} = \frac{(875 \times 7.5) + 2800}{.857 \times 12 \times 12 \times 10} = 7.6\ \#/\text{sq. in.}$$

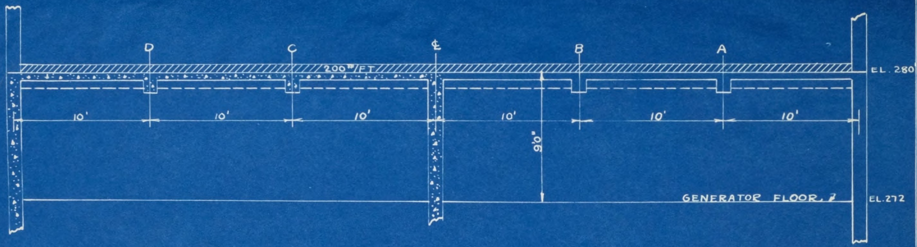


Fig. 2 - CROSS SECTION OF
POWER HOUSE OPERATING FLOOR ALONG ABCD (SEE Fig. 1)

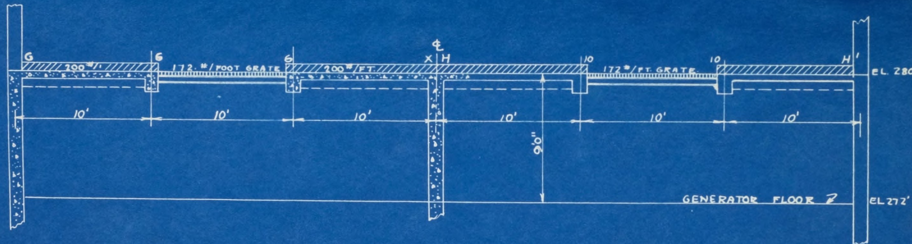


Fig. 33 - CROSS SECTION OF POWER HOUSE OPERATING FLOOR ALONG HH'GX (SEE Fig. 1)

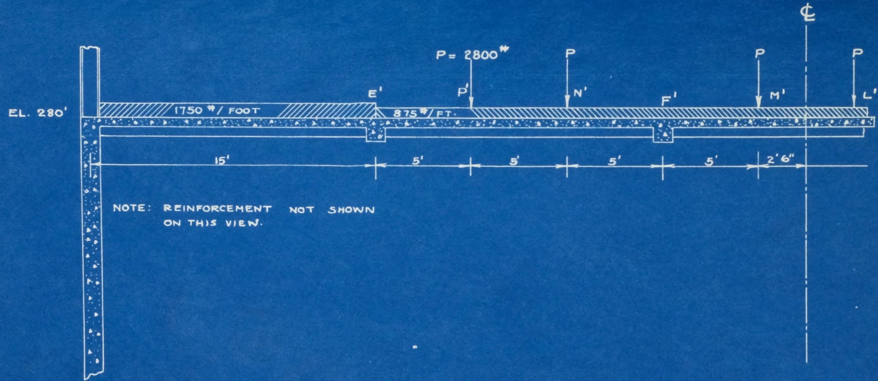


Fig. 34 - CROSS SECTION OF POWER HOUSE OPERATING FLOOR ALONG A A' (SEE Fig. 1)

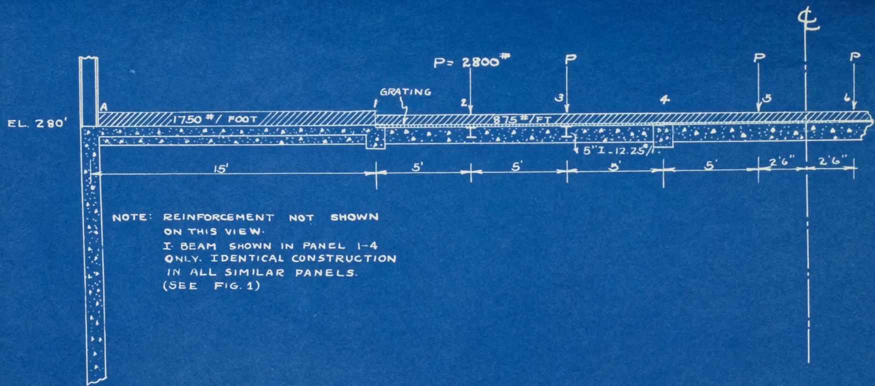


Fig. 35 CROSS SECTION OF
POWER HOUSE OPERATING FLOOR ALONG A-1-2-3-4 ETC. (SEE Fig. 1)

Fig. 35

DESIGN OF AUXILIARY FLOOR A B. (see elevation drawing) Fig. 37

The auxiliary floor, designed merely as a support for workers employed in the inspection of the turbines and other repair work, is not designed to carry excessive loads as have the floors in the preceding designs.

Loads on the auxiliary floor = 100# due to live load.
50# due to concrete.

For this design consider a section along AA'B'B. Using a 2000# concrete for which $f_c = 800$; $f_s = 16000$; $n = 15$; and the diagram on page 340 Hool Volume 1,

$K = 147$; $p = .0108$

$M = K\,bd^2$

$$M = \frac{w\,l^2}{10} = K\,bd^2.$$

$$\frac{200 \times 10 \times 10 \times 12}{10} = 147 \times 12 \times d^2$$

$d = 3.7''$ Use a 6 inch beam.

$A_s = pbd = .0108 \times 3.7 \times 12 = .48$ sq. inches.

For this steel requirement use ½" square bars at 6" c-c.

To prevent temperature changes from causing cracks and also to bind the entire structure together, 8 ½" bars will be placed transversly in each 15 foot panel.

The unit shearing stress $= v = \frac{V}{\Sigma o j d} = \frac{200 \times 10}{3.7 \times .857 \times 12 \times 2}$ equals 26.3 #/square inch. No diagonal reinforcement is necessary therefore.

The maximum bond stress occurs at the supports in the reinforcement for negative moment. At the supports theunit bond stress =

$$u = \frac{V}{\Sigma o j d} = \frac{200 \times 10 \times 6}{2(.857 \times 3.7 \times 12 \times 2)} = 79\#/\text{ sq. in. O.K.}$$

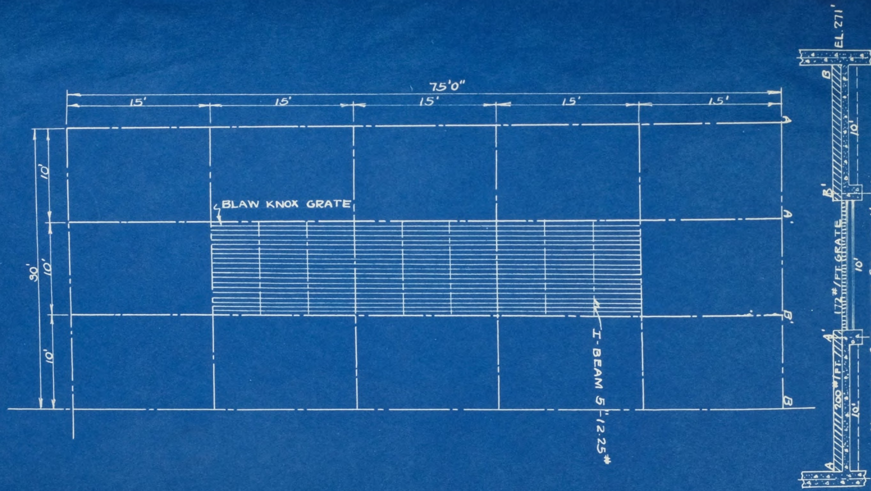


FIG. 37 "AUXILIARY" FLOOR PLAN

Fig. 37

The section across the grating is designed similar to the slab already designed and is made the same size as the floor just designed.

Design of Exterior Columns.

In ordinary column design, the loads are assumed to be concentric and applied at the longitudinal center of gravity axis of the column. Even in such a design, the assumption is necessary though the condition be theoretical only. However, when the load is considerably eccentric such as may occur in spandrel framing or as in this design, by beams carrying crane brackets, a column involved has to be carefully investigated for the combined stresses which result. Such a column must resist the sum of the direct and eccentric loads as well as the bending induced by the eccentric load.

If P = the direct load; R = the eccentric load; e = the eccentricity, or the distance of the point of application of R from the axis of the column-- then the direct stress equals

Direct stress $= \frac{P + R}{A}$, in which A equals the cross sectional area of the column.

The bending moment $= M_1 = R\,e$.

The stress due to this moment $= \frac{M\,c}{I}$.

The value of s is compression on the bracket side and tension on the exterior face.

$$\text{The maximum compression} = \frac{P + R}{A} + \frac{M\,c}{I}$$

$$= \frac{P + R}{A} + \frac{R\,e\,c}{I}$$

The loads are as follows:

Total dead panel load = 1300 #. x 8 = 10400 #.

Total wind panel load = 2200 #. x 8 = 22400 #/

Total snow panel load = 2800 #. x 8 = 8800 #.

The Total load is therefore: 41,600 pounds.

One half of the dead plus snow plus wind or P = 20800#. the total weight supported on theexterioe columns.

The load R is equal to the crane reaction which is equal to 72900 pounds. See page 411, Barrows, Water Power Engineering.

The distance "e" or the eccentricity equals 15 inches.

The length ofthe column is equal to 20 feet.

The section assumed consists of :

Web plate 12 x $\frac{3}{4}$"	A = 36.76 sq. in.
4 angles 6 x 4 x $\frac{3}{4}$"	I_{1-1} = 885"^{4}; r_{1-1} = 4.91"
	I_{2-2} = 266"^{4}; r_{2-2} = 2.69"

The total load is equal to 20800 + 72900 = 93700#.

$$P \text{ (direct)} = \frac{93700}{36.76} = 2550 \text{ pounds.}$$

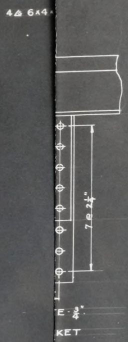
$$P \text{ (indirect)} = \frac{72900 \times 15 \times 6.25}{885} = 7710\#.$$

The total stress, both direct and indirect = 2550 + 7710 = 10620 Pounds, the actual stress.

The allowable stress = $19000 - 100 \times \frac{20 \times 12}{2.69} = 10750\#$.

The assumed section is alright.

The details ofthe column base, bracket and the column itself are shown on Sheet H, page 117c.



COLUMN BASE

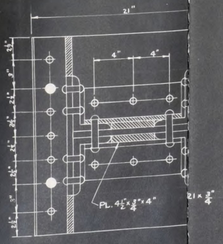


PLATE H

COLUMN DETAILS

Fig. 26

The design of the Column Base:

The base forthe column designed in the previous pages is a built-up base on concrete.

The loads on the column base equal 3000#, the weight of the column, in addition to the loads previouslu calculated.

The allowable pressure of the column on the concrete equals 500 pounds per square inch.

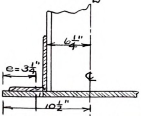
The required area $= \frac{97000}{500} =$ 194 square inches.

For this area try a base plate 20 inches square.

The actual area of pressure $= \frac{97000}{400} = 243$ square in.

Try a 6 x 4 inch shoe angles -

$M = \frac{p\,b^2}{2} = \frac{243 \times 3\frac{1}{4} \times 3\frac{1}{4}}{2} = 2{,}140$ #".



$t = \sqrt{\frac{6 \times 2140}{16000}} = .895$ " required.

$\frac{1}{2} + \frac{3}{4} = 1.25$ inches which is in excess of that required, but is used to give sufficient bearing to the steel column and still allow room for fastening the angles to the base.

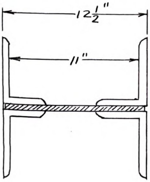
Use 6 x 4 x $\frac{1}{2}$" shoe angles and a 20 x 20 x $\frac{3}{4}$" plate.

Test the plate between the column flanges.

$M_e = \frac{p \cdot c^2}{12} = \frac{243 \times 11^2}{12} = 2450$ #".

The load outsidethe column face $= 3\frac{3}{4} \times 20 \times 2.43 =$

$= 18{,}220$#.



Single shear rivets, $\frac{3}{4}$", = 5300#.

There are 4 gauge lines available in each angle. Rivet the base as shown in the figure on Page .

Design of Counterforted Retaining Wall:

In the design of the counterforted wall as shown in the accompanying pages, the application of the pressure of the earth fill which forms a part of the earth dam, is made by a study of the earth pressure theories of Goodrich in the Transactions of the American Society of Civil Engineers, Volume 53, page 301.

Let P = the resultant earth pressure on a vertical surface for a length of wall equal to one foot.

h = total height of wall in feet.

w = equivalent liquid pressure.

ϕ = the angle of internal friction of the earth fill assumed to be 37°.

θ = the angle of inclination of the earth fill, equal to $26^{\circ}\ 40'$ as shown on the sketch.

$$\text{Then } P = \tfrac{1}{2}\, w\, h^2 \cos\theta \;\; \frac{\cos\theta \mp \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta \pm \sqrt{\cos^2\theta - \cos^2\phi}}$$

In this equation the negative sign in the denominator and the positive sign in the numerator give a value for the passive pressure. A reversal of the signs gives the value of the active pressure. The active pressure is the pressure used in the design,of the wall.

The load P acts at the third point of the height of the wall or 7 feet, ans is assumed to act at the same inclination as the earth fill of the dam.

$$P, \text{ the active pressure} = 19700 \; \frac{\sqrt{.89363 - .4}}{\sqrt{.89363 - .4}} = 7500\#$$

acting on a vertical surface. This is equivalent to a force of 6700# as the force P_H.

The stability of the counterforted wall as regards overturning may be determined in either of two ways, namely:

(1) By use of a theoretical formula for the thrust of the earth; or

(2) By making the stability against rotation equal to that of a gravity wall that is known to be safe. In this connection, attention is directed to the presence of a load of considerable magnitude, shown on thedrawing of the pressure distribution and labeled L. This force, which is the application of the roof loads and that portion of the floor loads which finds its way to the foundation, acts as a direct column load and tends to aid in preventing rotation. This force also tends to aid in causing a stable condition of the retaining wall. This load is considered to be composed of the entire roof load acting on the exterior column, and will be considered to be applied at the juncture of the counterfort with the mainwall.

The effect of sliding will not be considered due to the fact that the base of the wall, B C, on the drawing, is constructed monolithically with the floor of the power house.

The bearing power of the soil in short tons per sq. foot in this design is taken as 3 T. per square foot, and is considered to be a force at the toe of the base.

The type of wall selected in this design is a counterforted wall which was chosen for two reasons, (1) because it permits of the strongest pretection against the reaction

due to the fill of the earth being rolled and possible settlement of the earth dam, and (2) because the wall demands strengthening at the points of application of the superstructure loads; also an increase in size to permit fastening the shoe angle of the exterior column.

The principles recommended by the Joint Committee Report for the design of retaining walls are practically as follows:

(1) The unsupported toe and heel of the base slabs shall be considered as cantilever beams fixed at the edge of the support. (Conditions somewhat different in this particular case because of the monolithic construction of toe already mentioned.)

(2) The vertical wall of the cantilever shall be considered as a cantilever beam fixed at the top of the base.

(3) The vertical sections of counterforted walls and parts of base slabs supported by the counterforts shall be designed in accordance with the requirements for a continuous slab built to act integrally with restraining supports and assumed to carry uniformly distributed loads.

(4) Counterforts shall be designed in accordance with the requirements for T-beams in regard to flexure formulas and flange width. Stirrups shall be provided in the counterfort to take the reaction when the tension reinforcement of the base walls and heels of bases is designed to span between the counterforts. Stirrups shall be anchored as near the exposed face of the longitudinal wall and as close to the lower face as the requirements for protection covering permit.

(5) The shearing stress at the junction of the base with the counterforts shall not exceed the values specified for diagonal tension and shear in beams.

(6) Horizontal metal reinforcement shall be of such form and so distrdbuted as to develpp the required bond. To prevent temperature and shrinkage cracks in an exposed surface, not less than 0.25 square inches of horizontal metal reinforcement per foot of height shall be provided.

(7) Grooved lock joints shall be placed not over 60 feet apart to care for temperature changes.

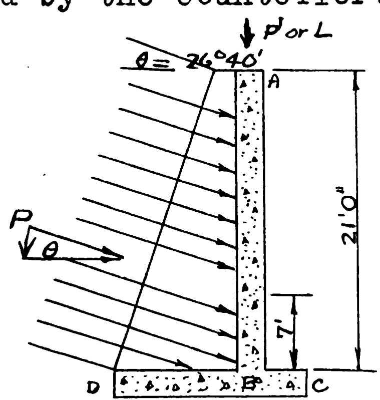
(8) The walls shall be cast as a unit between expansion joints, unless construction joints formed in accordance with the Joint Committee requirements are provided.

(9) Drains or "weep holes" not less than 4 inches in diameter and not more than 10 feet apart shall be provided. At leats one drain hole shall be provided in each pocket formed by a counterfort.

(10) The protective covering for the concrete in contact with the earth shall be 3 inches; that exposed to the weather shall be 2 inches.

DESIGN:

In this design the vertical slab is supported at intervals of 15 feet by vertical ribs or counterforts. These counterforts act as cantilevers and are securely tied to both the vertical wall and the footing. The projecting toe of the footing is a cantilever while the inner portion is a slab supported by the counterforts.



(1) B D is composed of narrow strips uniformly loaded with the dead weight of the slab, the downward weight of the earth, and the upward reaction of the soil.

(2) B C is considered as part of the floor system as already explained.

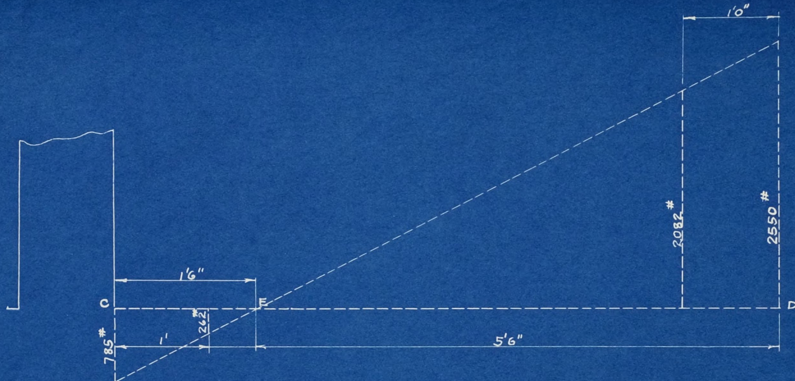
(3) The curtain wall is considered as made up of a series of horizontal strips and treated as slabs partly continuous and uniformly loaded. The pressure against this wall changes as the height of the wall so that the pressure upon different strips increases with the depth.

(4) The load on the counterforts is made up of that portion which is transmitted from the curtain wall, which takes the pressure. This value is obviously very small. The thickness of the counterforts is made sufficient to insure rigidity and give the necessary space for the reinforcing rods.

DESIGN OF THE WALL.

Referring to Page 126, Figure (a) the height of the wall is seen to be 21' 0". The counterforts are spaced at 15 feet center to center. The angle of inclination of the earth fill, equal to the angle formed by a 2:1 slope, is 26° 40'. The equivalent liquid pressure is assumed to be 25 pounds. The allowable pressure on the soil at the toe of thebase is assumed to be 6000 pounds per square foot. The coefficient of friction between the subgrade and the concrete base is assumed to be .4. f_s is equal to 16000 pounds per square inch; f_c = 650 pounds per square inch; "u" for deformed bars equals 100 pounds per square inch; for plain bars, "u" equals 80 pounds per square inch; The unit shear taken by the concrete equals 40 pounds per square inch.

Assume $\frac{x}{b}$ = .7 and investigate the toe unit pressure



(b)

PRESSURE DISTRIBUTION

and resistance to sliding. From Figure 29, Hool and Kinne, "Reinforced Concrete and Masonry Structures", when the ratio $\frac{x}{b} = .7$, $\frac{b}{h} = .475$, or b = .475 (21) = 10 feet, and x = .7 x 10 = 7'0". The unit pressure at the toe =

$$*p = 60 \left(h + 3h \frac{x}{b} + \frac{x^2}{.8b} \right)$$

= 4266 pounds per square foot.

This value is less than is allowable so is O. K.

For the purpose of investigating the resistance to sliding, it is assumed that the thickness of the vertical wall is .1 of the total width of the base. The thickness of the base may be assumed to be equal to the same proportion of the total height of the wall, namely, .1 x 21'. The difference between the weight of a counterfort and an equivalent volume of earth is so small it is neglected.

$$W = \left(0.1 \,(21(\ 10 + (.9)\ 21(.1)\ 10 \right) 150 + \left(.9\ (21)\ 7 + \frac{3.5}{2} (7) \right) 100.$$

W = 20,440 pounds plus the load L = 20440 + 3000 =

= 23440 pounds.

$$P_H = \frac{w'\ h^2}{2} = \frac{25\ (22)^2}{2} = 6050 \text{ pounds.}$$

Since P_H is less than .4 W, this base width and position of the vertical wall is satisfactory. These dimensions are: Base width = 10 feet; therefore thickness of vertical wall is 1 foot. The height of the vertical wall = 21 feet; therefore the thickness of the base is made 2'6".

The rear portion of the base slab is designed first as in the case of the cantilever wall. To get the downward pressure on this slab, assume that the thickness of the base slab = .1 the height of the vertical wall as before.

Then the downward pressure at C = .9 (21) 100 - .1 (21) 150 = 2200 pounds per square foot. The upward pressure at C = .7 (4266) = 2985 pounds per square foot. The downward pressure at C = 2200# and the upward pressure = 2985, then the resultant upward pressure at C = 2985-2200 = 785 pounds per square foot. The downward pressure at D = 2200 - 3.5 x 100 = 2550 pounds per square foot. Since the upward pressure at this point equals zero, then this value is the resultant downward pressure at this point. See the pressure distribution diagram (a).

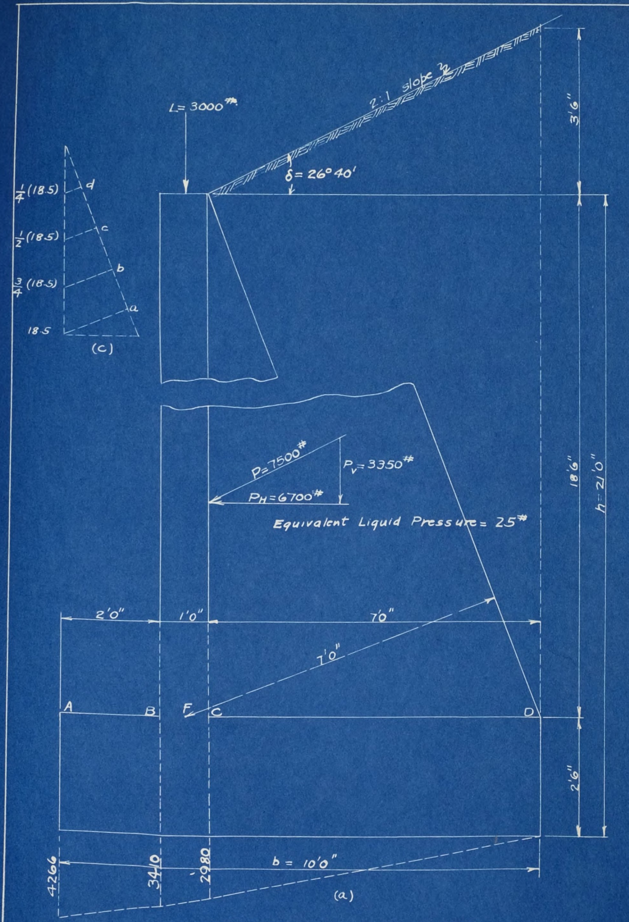
The portion of the slab C D subjected to the greatest bending moment is a strip one foot in width adjacent to point D. If it is assumed that the slab is continuous under the counterforts, and that therefore a moment coefficient of $\frac{1}{12}$ may be used, then the moment =

$$M = \frac{1}{12} w l^2 = \left(\frac{2082 + 2550}{2}\right) (5.5)^2 = 70{,}060 \text{ \# ''}.$$

From the formula $M = K b d^2$, $d = \sqrt{\frac{M}{K b}} = \sqrt{\frac{70060}{107 \times 12}} = 7.37''$.
The shear $V = \frac{w l}{2} = \frac{2082 + 2550}{2 \times 2} \times 8 = 9264$ pounds. For shear the value of $d = \frac{V}{b j v} = \frac{9264}{12 \times .863 \times 40} = 26.4$ inches.

A slab thickness of 2' 6" will be satisfactory. Substituting in the formula $A_s = \frac{70060}{16000 \times .863 \times 26.4} = .189$ sq. in.

Since the pressure at E is zero, the area of steel required at this point is zero, while at a point midway between E and D it is half of that required at D or .095 square inches. So for the 2 ¾ feet of slab adjacent to D, ½" round bars @ 6" c-c will be used, and in the remainder of the slab, ½" round bars @ 12" c-c will be used. This is more



PRESSURE DISTRIBUTION DIAGRAM.

steel than the design required but is put in to bind the base together. The bond stress in the bars near D =

$u = \frac{V\ b}{\phi} = \frac{40 \times 6}{1.57} = 153$ pounds per square foot. This value is slightly more than allowable butis used since twice as much steel is used as is necessary.

Since the slab C D is 2.5 feet thick, the clear height of the wall = 21 - 2.5 = 18.5 feet. The height of the earth fill acting on thevertical wall = 18.5 + 3.5 = 22 feet. There fore the pressure at the base of the wall is 22 x 25 = 550 pounds per square foot. Since thevertical wall is continuous across counterforts

$$M = \frac{1}{12} w\ l^2 \times 12 = 550 \times (5.5)^2 = 16620 \text{ inch pounds.}$$

The depth required for moment =

$$D = \sqrt{\frac{M}{K\ B}} = \sqrt{\frac{16620}{107 \times 12}} = 3.6 \text{ Inches.}$$

The shear $V = \frac{w\ l}{2} = \frac{550 \times 5.5}{2} = 1512$ pounds.

The depth required for shear = $d = \frac{V}{b\ j\ v} = \frac{1512}{12 \times .863 \times 40}$ equals 36 inches. Use a thickness of 12 Inches with an effective depth of 10½ inches.

The area of steel required at the base of the wall =

$$A_s = \frac{M}{f_s\ j\ d} = \frac{16620}{16000 \times .863 \times 10\frac{1}{2}} = .115 \text{ sq. in.}$$

Use ½" round bars at 6" c-c. This also is more steel than is needed. Place this steel 2" form the surface. $u = \frac{V\ b}{\Sigma 0}$ equals $\frac{40 \times 6}{.196} = 122$ pounds per square inch.

Use 4 @ 6"; 9 @ 8"; 12 @ 1' 0". See Figure (a).

The length of the toe A B is 10' - (7 + 1) = 2'0". The moment at B = $\frac{3410\ (2)^2}{2} + \frac{(4266+3410)\ 2^2}{3} = 7960$ #'.

The depth required for moment = $d = \sqrt{\frac{7960 \times 12}{107 \times 12}} = 8.6$ ".

The shear at B = $\frac{(4266 + 3410)}{2} \cdot 2$ = 7676 pounds. The depth required for shear = $d = \frac{V}{b\,j\,v} = \frac{7676}{12 \times .863 \times 40} = 20''$

The toe will be made 20" thick from B to A. The area of steel required is $A_s = \frac{7960 \times 12}{16000 \times .863 \times 20} = .342$ sq. in.

Use $\frac{1}{2}$" round bars @ 5.72 c-c. Then the bond stress =

$$u = \frac{V}{0jd} = \frac{7676}{\frac{12}{5.72} \times .863 \times 1.57 \times 20} = 133 \text{ pounds per sq. in.}$$

This value is more than allowable. The area provided by $\frac{1}{2}$" round bars @ 5" c-c is .47 square inches. Then the bond is 115 pounds which is still too large. The area provided by $\frac{1}{2}$" round bars at $4\frac{1}{2}$" c-c = .522 sq. inches. The bond provided by this steel is 105 pounds per square inch which is O. K.

The horizontal steel in the counterfort must be so designed as to be ableto withstand the horizontal pressure against thevertical wall. As given in the design of the verticall wall, the horizontal pressure on the lowest 1' strip of vertical wall is 550 pounds per square foot.

Since the spacing of the counterforts is 15' c-c, the total force is 15 (550) = 8250 pounds. The area of steel required = $\frac{8250}{16000}$ = .515 square inches. This area may be provided by $\frac{1}{2}$" square bars @ 7" c-c.

The center of the strip 1' wide where the pressure is 550 pounds per square foot is 18' 0". The area provided by $\frac{1}{2}$" round bars @ 12" c-c is .20 square inches. This spacing may be started ata point $\left(\frac{.2}{275}\right)$ 18 = 13' below the top of the vertical wall. Since this spacing of rods is practically the same as the spacing of the horizontal rods in the front vertical slab, the same spacing

will be adopted for both.

The total downward force to be resisted in the outermost section of the counterfort = $15 \left(\frac{2082 - 2550}{2} \right) =$ 34740 pounds. To carry this, an area of steel of $\frac{34740}{16000}$ or 2.17 square inches must be provided. This area can be furnished by 3 $\frac{3}{4}$" bars @ 6" c-c. This spacing is also adopted for 4' of the counterfort adjacent to D and will be increased to 12" c-c. for the next 4 feet. From E to C no vertical rods are needed since the resultant pressure is upward.

Both the horizontal and vertical rods in the counterfort are hooked around the horizontal bars in the vertical slab and the back of the base slab.

The amount of steel required along the inclined edge of the counterfort is determined by taking moments around point F. The perpendicular distance form F to the edge of the counterfort is 7'0". Allowing 3" for a protective coating, the effective depth = 6' 9" = 81 inches. The height of earth acting on the vertical wall is 24' 6" as determined in this design of thewall. Then the Bending Moment is

$$M = \frac{w' \ h^2}{2} \cdot \frac{h}{3} \cdot 12 = \frac{25 \times 24.5 \times 12}{6} = 736000 \text{ "\#.}$$

per foot of wall or 15 x 736000 or 11,040,000 "# per counterfort. The area of steel required is $A_s = \frac{M}{f_s jd} = \frac{11040000}{16000 \times \frac{7}{8} \times 81}$ equals 9.75 square inches. The area of steel required at various points is directly preportional to the cube of the height butis inversly proportional to the effective depth. Sincetheeffective depth is directly proportional to the height, the result is to make the area of steel required

directly proportional to the square of the height. If 9.75 square inches is needed at a point labeled "a" , then the steel needed at points b, c, and d respectively is as follows.

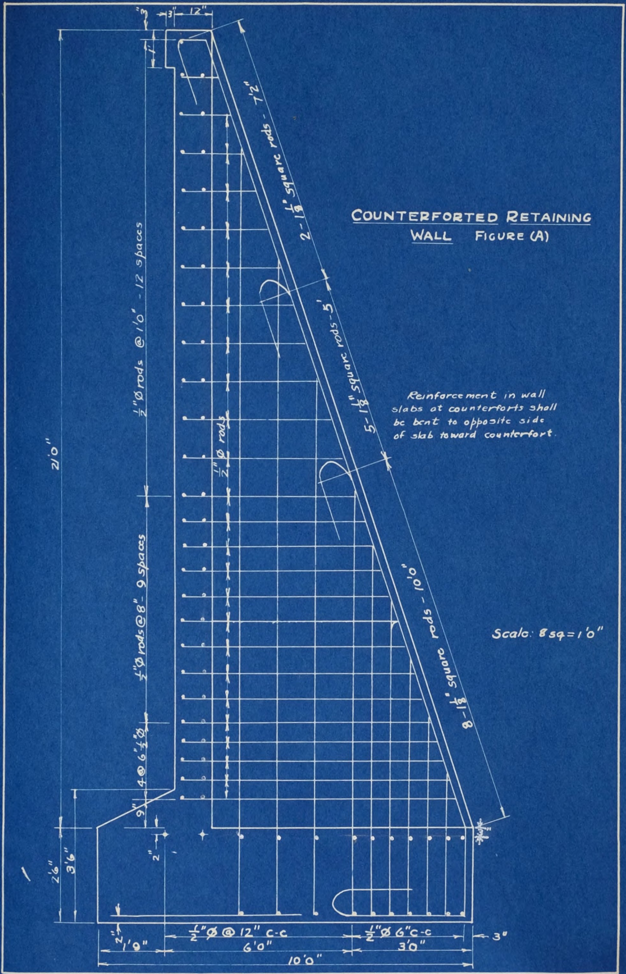
(b)-$(\frac{3}{4})^2$ x 9.75 = 5.5 square inches.

(c)-$(\frac{1}{2})^2$ x 9.75 = 2.44 square inches.

(d)-$(\frac{1}{4})^2$ x 9.75 = .61 square inches.

For this steel requirement use $1\frac{1}{8}$ inch bars, 8 at a; 5 at b; 2 at c; and 2 at d.

The rods are extended beyond the theoretical distance a short way to care for bond. Additional strength is gained by providing hooks at the ends as shown in Figure A.



COUNTERFORTED RETAINING WALL FIGURE (A)
Reinforcement in wall slabs at counterforts shall be bent to opposite side of slab toward counterfort.
Scale: 8 sq = 1'0"
2-1⅛" square rods - 7'2"
5-1⅛" square rods - 5'
8-1⅛" square rods - 10'0"
½" Ø rods @ 1'0" - 12 spaces
½" Ø rods @ 8" - 9 spaces
4 @ 6" ½" Ø
½" Ø rods
21'0"
2'6"
3'6"
½" Ø @ 12" c-c
½" Ø 6" c-c
1'9"
6'0"
3'0"
10'0"
3"
12"

Design of Stairways:

The stairways in the power house are of reinforced concrete construction and the details are shown on Figure , a sectional view of the power house.

For purposes of design, the span of the slab which composes the stair is assumed to be equal to the horizontal projection of the length of the slab between two floors, and the design is considered as if it were a straight slab.

The live load on the stairs is taken as 100# per square foot of the horizontal projection of the slab. The unit dead load of the stair portion is figured as follows:

$$\text{Unit dead load} = \frac{(14 \times \frac{1}{2} \times 4 \times 150) + 15(\frac{10}{12} \times \frac{7}{12} \times \frac{1}{2} \times 4 \times 150)}{12\frac{1}{2} \times 4}$$

= 128 # dead load.

Live load = 100 #

228 # TOTAL LOAD.

Since the stairs are poured separately from the rest of the building and are joined to the floor only be stair dowels the slab is considered as simply supported, using $M = \frac{wl^2}{8}$

$$M = \frac{228 \times 12.5 \times 12.5 \times 12}{8} = 53,500 \text{ inch pounds.}$$

Then:

d = depth of slab = $.29\ C\sqrt{M}$ or $C_1\sqrt{M}$

and

$A_s = pd$ per foot of width.

Use 1:2:4 concrete for which ultimate strength = 2000#; $n = 15$; $f_s = 16000$; $f_c = 800$; $k = .429$; $C = .083$; $C_1 = .024$; $R = 147$.

In the above formulae, the constant C or C_1 is a constant

which is based simply on the values taken for a givenconcrete and which may be referred to in the volume "Concrete, Plain and Reinforced", by Taylor, Thompson, and Smulski, Page 205.

The width of tread chosen for the stairs in a commercial building such as a power house is 10 inches and the dimension ofthe riser is found by a simple computation involving a division of the distance between floors by the number of steps desired thus,

Riser $= \frac{12 \times 9}{15} = 7.2$ inches.

The width of the stairs is taken as four feet. There will be little or no danger of congestion on the stairs of the power house since they are for the purpose of gaining access to the auxiliary floors only for repair of turbine, etc.

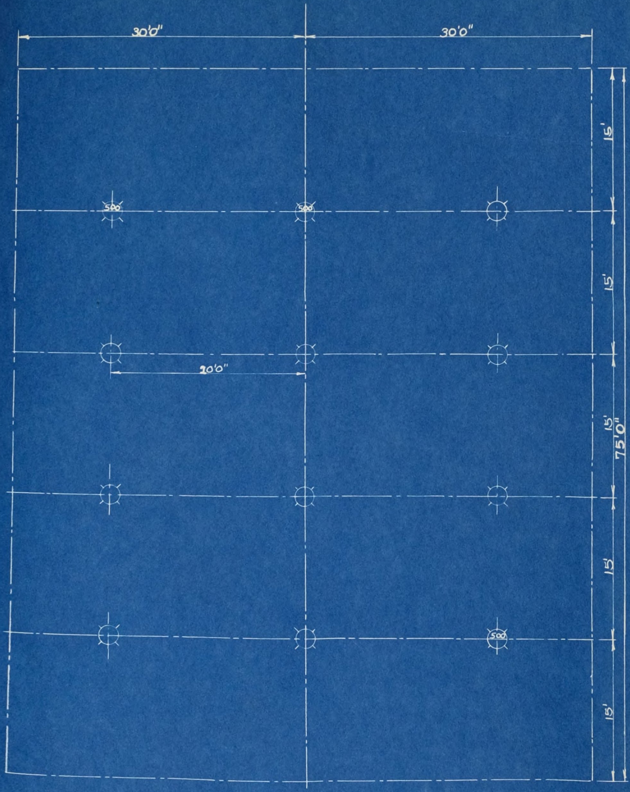
Applying the formula on the opposite page,

$d = .024\sqrt{53500} = 5.5"$ plus one inch protective coating. This makes the total depth equal to 6".

$A_s = 5.5 \times .0107 = .05885$ sq. in. per inch.

The spacing of $\frac{5}{8}"$ round bars $= \frac{.3068}{.05885} = 5.2$ inches. Space 5 inches center to center.

Cross reinforcement is placed in each riser to prevent temperature cracks and will be a $\frac{3}{8}"$ bar.



FLOOR PLAN OF POWER HOUSE

LIGHTING PLAN

FIG. 38

Fig

Specifications for Truscon Industrial Door for Power House.

General:

All doors of 10' or over in size shall be Industrial Doors of the heavy tubular type as shown on the detail drawing of the swing unit installed in the power house. See Figure .

Material:

All stiles and rails shall be constructed from cold-rolled welded steel tubing.

All windows included in doors shall be constructed from hot-rolled new billet steel.

Construction:

The stiles, top rail, cross rails and bottom rail shall be constructed of No. 13 gauge cold-rolled welded steel tubing, 4" x 2½".

The corner shall be mitered and internally reinforced, the reinforcing extending 10" in both directions from the corners. All miter joints shall be welded and ground smooth.

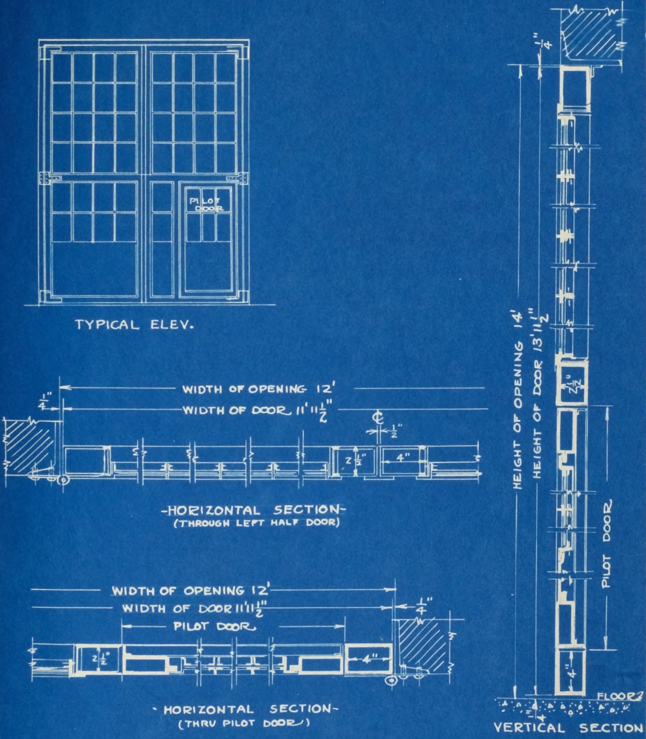
The lower portion of the doors shall be fitted with not less than No. 16 gauge steel panel bolted in place.

The upper portion of the door shall be fitted with window built up by Truscon standard members and glazed with glass lights as shown on the drawing. The glass shall be held in place with putty and glazing angles.

Frames:

Where shown on the drawings, steel channel frames for all door openings shall be furnished and installed by the contractor supplying the structural steel.

Hardware:



POWER HOUSE DOOR DETAILS FIG. 39

Fig. 39

Hardware:

Sliding doors shall be hung from Truscon Standard double trolleys and heavy channel track and shall be equipped with flange guides, back stops, etc.

Swinging doors shall be equipped with Truscon Standard mortise cylinder locks (or lever latch and padlock brackets) and heavy handles on inside and out.

Where doors are hung in pairs, one leaf shall be equipped with Truscon Standard foot bolt and chain bolt.

An astragal shall be provided by door manufacturer with all double swing or double slide doors.

Painting:

All doors shall receive one brush coat of red oxide of iron paint before shipment.

Erection:

The erection of doors furnished in combination with steel window contracts shall be handled by the manufacturer of the same.

Specifications for Truscon Pressed Steel Frames for use in Power House.

General:

All frames unless otherwise specified shall be pressed steel frames as manufactured by the Truscon Company.

Material:

Steel sheets used in the manufacture of frames shall be cold rolled No. 11 gauge, full pickled, re-annealed steel of U. S. Standard gauge and patent leveled.

Construction:

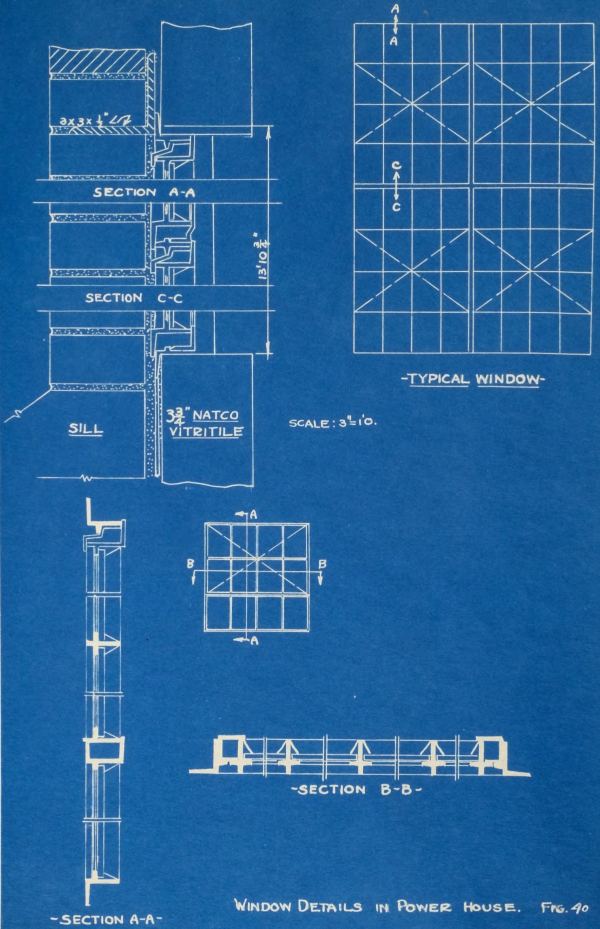
All shop joints shall be continuous welded and ground smooth.

Where horizontal field splices are necessary, they shall be made above or below a horizontal mullion. An inside splice fitting the vertical mullion shall be shop welded on one section, the splice to project out, allowing the other section to be driven over the splice form with a driving fit.

Where vertical field splices are required the entire frame is spliced along a single vertical line the center line of a vertical mullion. The vertical mullion is split and each half is shop welded to the head, sill or horizontal mullion part or parts shipped with it. Mullion cover plates join the two sections of the split vertical mullion in the field. This arrangement of splices allows all corners of the pressed steel frame to be shop welded.

Head and jambs shall be formed with fin for anchoring into the masonry.

Where size of frame will permit shipping as a unit,



WINDOW DETAILS IN POWER HOUSE. FIG. 40

vertical mullions shall be welded throughout.

Painting:

All frames shall be given a coat of protective paint before shipment.

Erection:

The erection of frames shall be handled by the manufacturer of the same.

For details of frame construction refer to Figure p 139.

REFERENCE LIST

Barrows, Water Power Engineering. McGraw-Hill Book Company 1927.

Creager and Justin, Hydro-Electric Handbook. John Wiley and Sons, Inc. 1927.

Hool and Johnson, Concrete Engineers' Handbook. McGraw-Hill Book Company 1918.

Hool and Kinne, Reinforced Concrete and Masonry Structures, McGraw-Hill Book Company 1924.

Hool, Reinforced Concrete Construction, Volume 1. McGraw-Hill Book Company 1927.

Taylor, Thompson, and Smulski, Concrete, Plain and Reinforced Volume 1. John Wiley and Sons, Inc. 1925.

Turneaure and Maurer, Principles of Reinforced Concrete Construction, Volume 1. John Wiley and Sons, Inc. 1932.

Urquhart and O'Rourke, Design of Steel Structures. McGraw-Hill Book Company 1930.

Voss and Varney, Architectural Construction, Volume 2, Book 2. John Wiley and Sons, Inc. 1927.

The Michigan Engineer, October 1928.

Power, September 1928.

Engineering News Record, Volume 69, 1914.

Pocket has: 1 Plate