REPORT OF COMMITTEE ON STRESS DISTRIBUTION IN EARTH MASSES

By Dr. D. P. Krynine, *Chairman*,
Yale University

This year the activities of the Committee were concerned with further collecting and systematization of materials for a Manual for highway and runway engineers, in the province of earth engineering. In this connection, Messrs. Jacob Feld and Robert G. Hennes have prepared the following papers.

ABUTMENTS FOR SMALL HIGHWAY BRIDGES

Part 2

By Dr. Jacob Feld, *Consulting Engineer*,
New York City

SYNOPSIS

This report covers parts "B" (Materials and Stresses) and "C" (Types of Abutments—Methods of Design) of the general outline of all factors relating to the design of abutments for small highway bridges.

Under "Materials and Stresses" are listed the applicable characteristics of various types of Masonry, (dry and mortar rubble, ashlar, brick, mass concrete and precast units), Reinforced Concrete, Concrete sheeting and piles, Steel sheeting and piles, Cribbing and Arch structure materials. Specification references, special precautions and practical recommendations are listed for each material. Typical uses are illustrated by photographs and diagrams.

Under Types of Abutments—Methods of Design, gravity walls, cribbing, counterforted walls, cantilever walls, sheeting, open frames, hollow boxes, filled boxes, anchored bulkheads and braced bulkheads are described as types with variations encountered, with outlines of the method of analysis. Special formulas are derived for the unusual types and recommendations made for approximate and accurate design solutions. Illustrations by photographs of actual designs are given for most of the types. Further reports to complete the outline published in 1943 are planned for succeeding years.

This report is the continuation of the report published in the *Proceedings*, Highway Research Board, Vol. 23, page 403 (1943). It deals with the sections of the general outline: B. *Materials* and *Stresses*, C. *Types* of *Abutments—Methods* of *Design*.

An attempt has been made to overcome the impact of habit and of propaganda advertising and to present for consideration all economically possible materials and types of abutment design. Economy of materials varies with the locality. Availability does not necessarily mean economy, and must be considered in conjunction with the availability and cost of correspondingly necessary labor and also the local history of useful life. The cost of an abutment is the ultimate cost, including the initial cost, maintenance during the period of expected life of the bridge structure, salvage value and demolition cost at the end of the expected life period. Prevalence of any one material or any one type in a locality must not be taken as prima-facie evidence of the economy of such material or type. Often, the proximity of a source of supply, or of a specialized construction firm, results in the establishment of precedence, remaining long after the original causes have ceased to exist. Many are the examples which can be cited of delayed acceptance of new material uses and of design methods, not restricted to any part of the nation, which hamper proper progress in the use of engineering development for the benefit of mankind. Combinations of materials are often advisable and may prove more economical than the simpler designs of single materials. Figure 1, shows an example of stone masonry, concrete, timber sheeting, wood piles and steel beam combination, which

is exhibited not as a recommendation, but as an excess of "experimental design."

B. MATERIALS AND STRESSES

General and detailed specifications for the various materials are found in the Standard Specifications of the American Association of State Highway Officials and of the American Society for Testing Materials.

(a) *Mass Masonry*

Mass masonry, in the form of natural or man shaped units laid in mortar or assembled in stable shape without filled joints is the oldest material used for bridge abutments. Its long life with practically no maintenance cost and the salvage value of the masonry units seem to have been forgotten. The recent tendency to cover concrete abutments

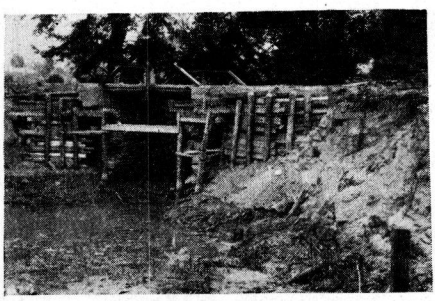

Figure 1. Versatile Combination of Materials (Not Recommended). Courtesy of Toncan Culvert Mft. Assn.

with cut stone (for "architectural reasons") seems to be a re-discovery of formerly known properties of this material, pleasing appearance and long life. Mass masonry is used in gravity type walls and abutments and may be of the following classes:

Dry Rubble is made of coursed, random and random range work, rough squared and sometimes dressed, laid in natural beds with open joints chinked when not in good contact. Such masonry should not be subjected to more than 150 lb. per sq. in. maximum compression. Figure 2 is an example of the use of local stone from fence walls in economical piers and abutments. This structure was built 20 years ago with semi-skilled farm labor, with no further expenditure since its original construction cost.

Mortar Rubble is made of similar materials as dry rubble, but with all joints filled with cement mortar. The mortar must be plastic, using hydrated lime, natural cement or masonry mortar admixture to provide proper workability and to reduce shrinkage. The mortar strength would be not less than 500 lb. per sq. in. in 7-day 2-in. cube tests. Such

Figure 2. Rubble Masonry

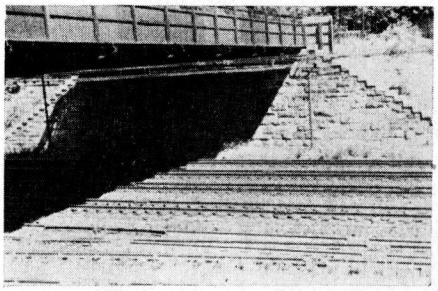

Figure 3. Cut Stone Masonry

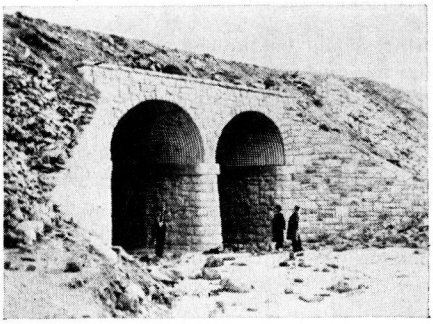

Figure 4. Split Stone Masonry. Courtesy of Toncan Culvert Mft. Assn.

masonry should not be subjected to more than 250 lb. per sq. in. maximum compression and to not less than 20 lb. per sq. in. minimum compression. Exposed joints should be tooled out 1 in. deep and filled with a low shrinkage mortar.

Ashlar masonry is made of cut stone masonry with the individual stones dressed or tooled to dimension and all joints filled with cement mortar. A typical example of coursed masonry is Figure 3, and a variety of jointing is shown in Figure 4. The mortar should be plastic and made of ingredients which will not react chemically with the stone or leach out after exposure. Exposed joints should be tooled out 1 in. deep and caulked with a low shrinkage mortar colored to match the stone or as desired. The mortar strength should be not less than 750 lb. per sq. in. in 7-day 2-in. cube tests and 1500 lb. at 28-day age. Such masonry should not be subjected to more than 500 lb. per sq. in. maximum compression and must not be subjected to any computed tensile or zero stress at any section.

Brick masonry is made of burned clay units laid in cement mortar with filled joints. Mortar should be the same as described for ashlar masonry except that exposed joints can be finished flush as the courses are laid up. Permissible stresses are the same as for the ashlar masonry. Reinforced brickwork is not often used in the U. S. A. although very common in India and also used to some extent in the Near East. The introduction of reinforcing in the mortar joints permits tensile stresses, but the economy of the design is seriously affected by labor availability and the cost of handling such small quantities of reinforcement.

Mass concrete is made of low cement content mixes, often with stone "plums" introduced while the mix is still plastic. Such masonry should not be subjected to more than 0.45 of the 28-day cylinder strength as a maximum compression load and to no tensile or zero stresses at any section. Pleasing mass with simple details is shown in the type exhibited in Figure 5.

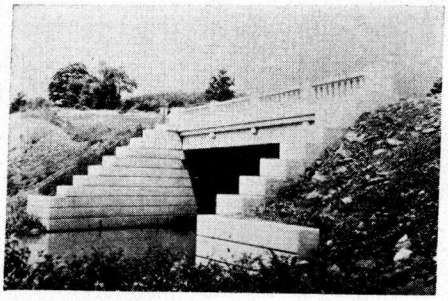

Figure 5. Mass Concrete. Courtesy of Portland Cement Assn.

Masonry units either of natural stone or precast concrete can be used as revetment surfacing to protect bank slopes and thereby protect a shallow abutment wall and eliminate the necessity for wing walls. Figure 6 shows masonry rip rap protection of the slopes and of the shallow bridge abutment. Another use, to protect pile abutments, is shown in Figure 24.

Concrete Faced Dry Masonry. In small bridge work, in private recreational camp areas, the writer has used a combination of concrete facing and field stone body for abutments, which has proved very economical. Using a single form set about 2 in. in front of a dry laid (reclaimed fence wall) stone masonry, a fluid mix of concrete containing $\frac{3}{4}$-in. maximum gravel is poured against the form, partly filling the voids in the stone masonry. Exterior vibration of the form produces a satisfactory surface. Similar concrete is poured on top of all exposed masonry and broomed in. Since the greater stresses occur at the face of the abutment, where the masonry is bonded by the concrete, somewhat higher stresses can be permitted than for dry masonry. The dry masonry at the rear of the wall also acts as a drainage aid when the backfill materials are slow draining.

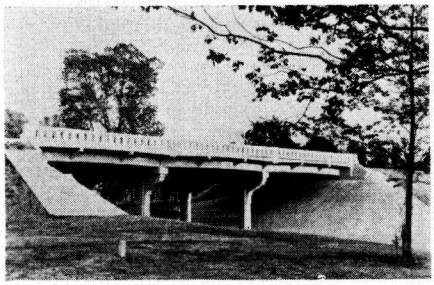

Figure 6. Precast Concrete Slope Protection. Courtesy of Portland Cement Assn.

General Note for all Mass Masonry Abutments. Sudden variation in height or thickness must be avoided, otherwise unsightly and possibly unsafe cracks will occur. Variations in height or thickness should not exceed 10 per cent of the larger height or thickness at any change in section and further reductions should be made in steps or stages not less than 5 ft. apart.

(b) *Concrete*

Reinforced concrete should be designed as provided in the A.A.S.H.O. Standard Specifications,[1] Section 3.4.11. The concrete may be of the following classes (as in A.A.S.H.O. specifications).

Class	28-Day Test	Aggregate	Use
A	3000	$1\frac{1}{4}$-in. max.	General and where exposed to water
B	2200	2-in.	Massive and lightly reinforced
C	1500	$2\frac{1}{2}$-in.	Massive unreinforced
X	3000	2-in.	Special reinforced sections if massive
Y	3000	$\frac{1}{2}$-in.	Hand railings, etc.

If concrete is deposited in water, use Class A with the cement content increased 10 per cent.

Permitted stresses should conform to the A.A.S.H.O. design specifications.

Form work should be designed to give pleasing and durable concrete surfaces. Impervious forms do not permit the escape of air and water bubbles before the concrete has set and result in a smooth but pock-marked surface. Poorly built forms permit the loss of mortar and fines and result in honeycomb exposures. The added cost of absorptive forms is little if proper allowance is made for obviating the finishing of the surface. No patching or cement wash procedure will correct or eliminate poor concrete surface. Ornamental details should always be recesses in the concrete, and designed for ease in form construction and especially for ease in form removal. Freedom of design is possible with concrete since it has no limitation of manufacturing standards as exists with other materials. Figure 7 is an example of curved wing walls and the possibility of simple designs is shown in Figure 8.

Deterioration of the surface from weathering is reduced by proper concrete mix proportioning, avoidance of aggregates which have poor bonding characteristics and by surface curing. The use of sulphate resistant cements and possibly also of resinous admixtures in the cement should be considered. Temperature volume changes and shrinkage are resisted by reinforcement. At present, there is no definite method for the adequate design of such reinforcement. The customary empirical rule is $\frac{1}{4}$ per cent of the cross-sectional concrete area in the horizontal direction, with a lesser amount vertically. Since shrinkage cracks almost always open in vertical directions, the reduction of the vertical steel to the minimum construction requirements of spacer and tie bars may be justified.

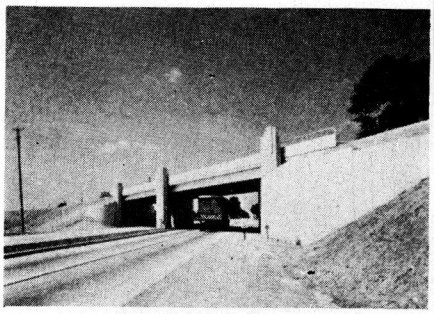

Figure 7. Curved Concrete Wing Walls. Courtesy of Portland Cement Assn.

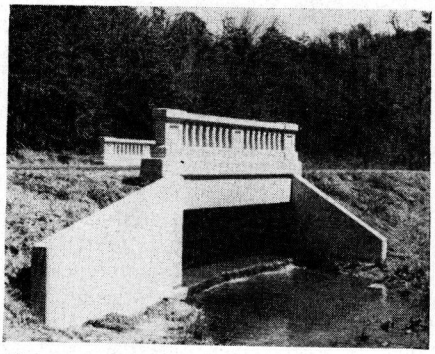

Figure 8. Reinforced Concrete. Courtesy of Portland Cement Assn.

(c) *Sheeting*

Timber Sheeting and *Piles* except in very minor or temporary structures should be treated with a preservative suitable for the local climatic characteristics. Timber and allowable stresses shall conform to the A.A.S.H.O. Spec. 2.2.2. and the preservative treatment to Spec. 2.21.2. Timber sheeting is usually employed in combination with timber anchorage or bracing piles. A further development of combining vertical piles for the

[1] Standard Specifications for Highway Bridges, Third Edition, American Association of State Highway Officials.

support of bridge loads and batter piles for bracing and resistance of the lateral loads is shown in Figure 9.

Concrete Sheeting and *Piles* should conform to the A.A.S.H.O. Spec. 2.2.3. for concrete sheet piles. Reinforcing should have at least 2 in. of covering at all faces. Since sheeting is to be exposed to impact stresses during driving, of uncertain values but probably far in excess of normal loading stresses, proper curing and age of the concrete must be assured. Fast setting concrete normally tends to be brittle, and slow setting cements should be specified. When exposed to salt water, special studies of the concrete mix to obtain maximum density are also necessary. When sheeting in place is exposed to large temperature variation, aggregates which have large coefficients of temperature expansion and

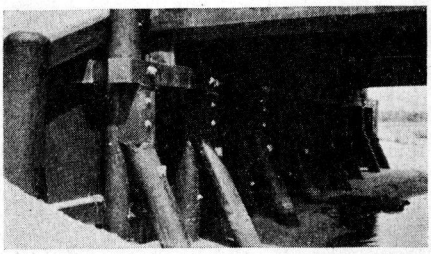

Figure 9. Batter Pile Braced Timber Sheeting. Courtesy of Amer. Wood Preservers Assn.

those which have unequal expansion coefficients along crystal axes must be avoided.

Steel Sheeting and *Pile* materials have definite structural characteristics. Unsatisfactory results are always traced to improper construction methods. Bearing values for vertical loads can best be predicted by the use of friction resistances over imbedded surfaces rather than from any driving formula. The lateral resistances of sheeting are determined by static methods on the assumption of full interlock values, unless there is positive proof of interlock failure (Fig. 10). Care in driving, and relief of obstructions by jetting and of interlock friction by grease, with progressive driving of adjacent sheets, all may somewhat slow up the work, but will prevent curling of the sheets and interlock failure. Where water seepage through the joints is not desired, an injection of heavy grease or colloidal clay (bentonite or celite) by pressure guns into the interlock spaces will be found a simple cure. On one operation, gas tank residue tar forced into the interlock by an alemite gun with a tube extension almost 30 ft. long, sealed the sheeting joints sufficiently to eliminate all leakage under almost 20 ft. of exposed water head. Loss of metal thickness from rusting is almost negligible, but where stray electric currents are to be expected, weld bonds across the interlocks and avoidance of acid producing fills, such as cinders must be specified.

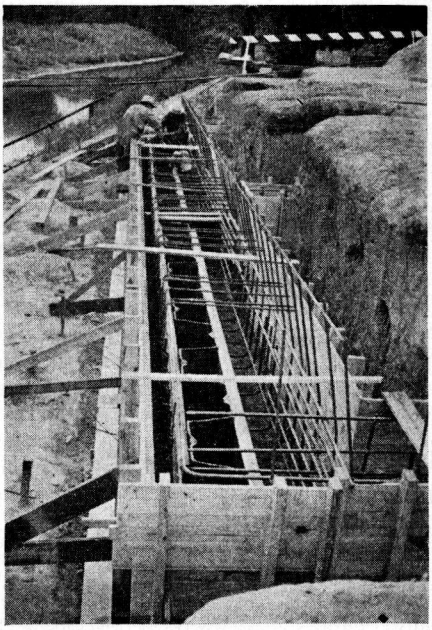

Figure 10. Steel Sheet Piles with Concrete Cap. Courtesy of Carnegie Illinois Steel Co.

Steel H sections used as piles for vertical and lateral loads are becoming more popular. In shallow rock locations, piles are driven into the rock for supporting loads economically and to avoid dependence upon frictional resistance. To increase the bearing value and also lateral stability, H piles are sometimes set into blown out pipe sections filled with concrete (Fig. 12).

Cantilever sheeting abutments are often increased in strength by using the bridge structures as a top support, as is shown in Figure 13, where steel columns are also used to brace sheetpile sections.

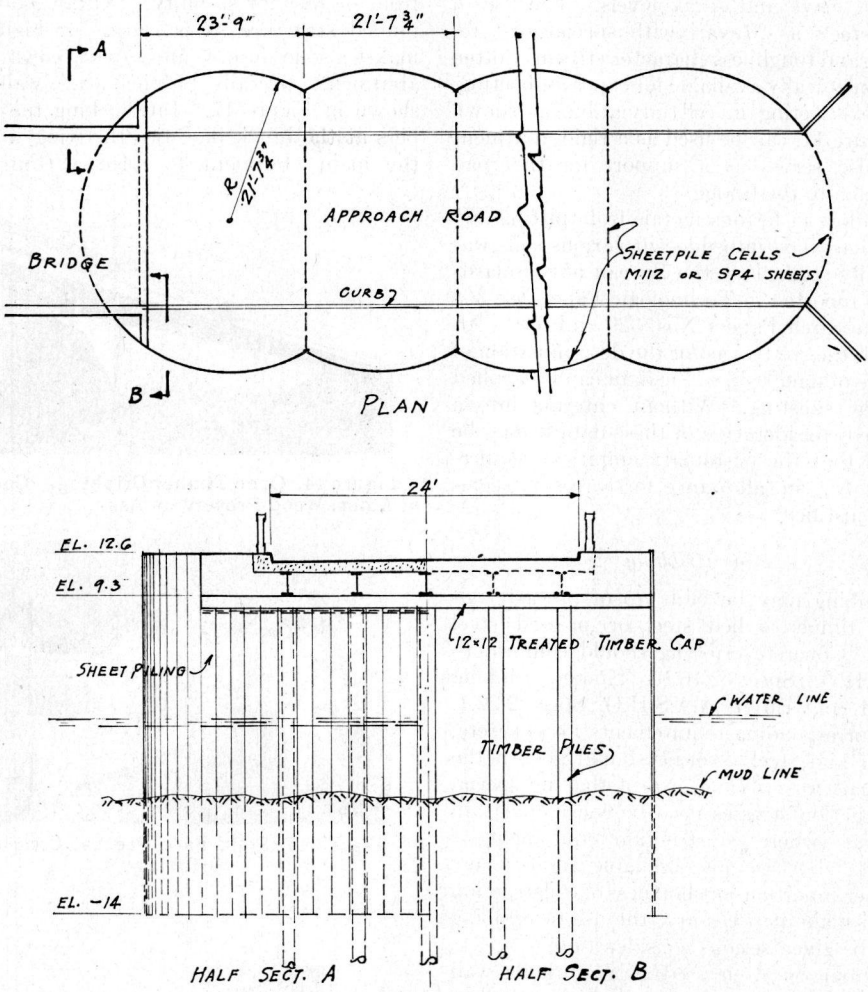

Figure 11. Steel Sheetpiling Filled Cell Abutment Apalachicola Bay Bridge. Designed by Florida State Road Department

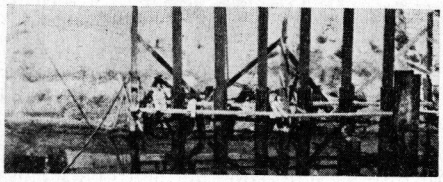

Figure 12. Steel H. Piles in Tubes. Courtesy of Carnegie Illinois Steel Corp.

When directly exposed to tidal waters, the face of the steel should be coated with tar or asphaltum paints, especially within tidal range and for a height sufficient to include

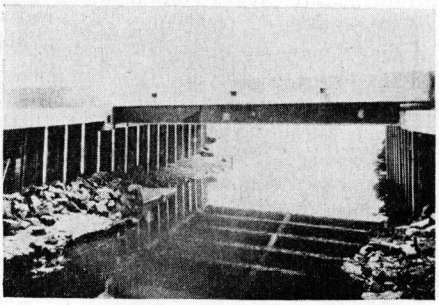

Figure 13. Steel H Piles Bracing Sheeting. Courtesy of Carnegie Illinois Steel Corp.

highest wave and spray levels. The use of such steels as Mayari with special rust resisting and toughness characteristics may often be economically available for exposed sheeting.

Steel sheeting in cell formation, as shown in Figure 11, can be used as a rigid abutment and also serves as a support for the road approach to the bridge.

The loss in ferrous metals from pitting and corrosion when imbedded in various soils was carefully studied by the Bureau of Standards and is reported in Technologic Paper No. 368 and Research Papers Nos. 329 and 638. Although the study was for the determination of the life of metal pipes, the data can be applied to steel sheeting. Without entering into a detailed consideration of these data it may be stated that the customary empirical requirement of $\frac{1}{16}$ in. allowance for corrosion seems to be justified.

(d) *Cribbing*

Cribbing may be built up of precast concrete, timber, rolled steel or pressed steel units. Concrete cribbing should conform to A.S.S.H.O. Spec. 2.16.1. Timber cribbing should conform to A.A.S.H.O. Spec. 2.22.1. The corresponding requirements for concrete, timber and steel sheeting should govern the materials for cribbing, except that no special construction stresses need be considered. In locations where construction equipment or skilled labor is not available, and where weather conditions or shortness of construction season make it necessary, the use of cribbing must be given serious consideration.

Cribbing is often used as a retaining wall for a high slope at the top of which a shallow masonry abutment is built for the bridge span; such combinations may not make the prettiest pictures, but are often the most economical solution of a problem.

Cribbing may be used with open or with solid face exposures. Stability is obtained by either interlocking headers with stretchers or by pinning the buried units to the face cribbing. The high salvage value of all cribbing types must be evaluated in estimating comparative abutment costs. Figure 14 is an example of open timber cribbing. An example of solid face concrete crib with pin interlocking members is shown in Figure 15.

Timber crib members can also be set up as a laminated facing with piles on either the front or rear for stability. A fully bolted or pinned structure as is shown in Figure 16 makes a solid durable and rapidly constructed abutment. Details of the wing walls are shown in Figure 17. Interlocking the members at the break line, ties the wing walls to the main abutment to form a continuous structure as shown in the high abutment in Figure 18.

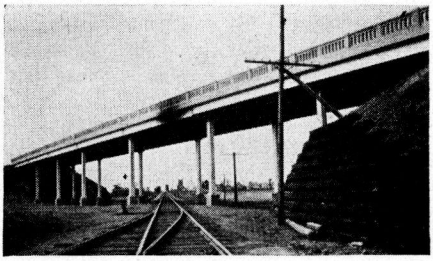

Figure 14. Open Timber Cribbing. Courtesy of Amer. Wood Preservers Assn.

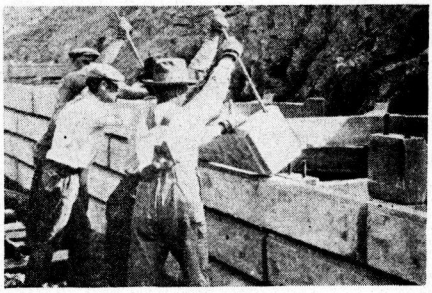

Figure 15. Solid Face Precast Concrete Cribbing

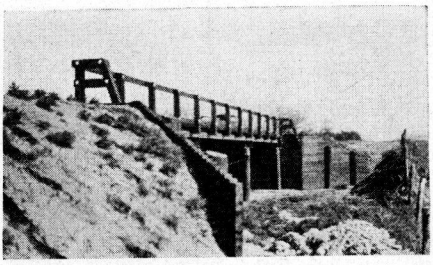

Figure 16. Pile Braced Laminated Timber. Courtesy of Amer. Wood Preservers Assn.

There are numerous patented details of shapes and interlocks for precast concrete cribbing. An early type is shown in *Engineering & Contracting*, Oct. 1924, p. 961, showing rectangular log stretchers set between I shaped

headers in open box form. Another early type used H section stretchers set with webs horizontal and with formed lugs on the ends of square section headers.

(e) Frames and Arches

In rigid frames of concrete and steel combinations, the abutment is part of the bridge design. Recent studies by the writer of a steel-concrete rigid frame consisting of imbedded structural steel ribs spaced about 2 ft. apart and designed as a rigid frame to support the weight of the wet concrete and forms,

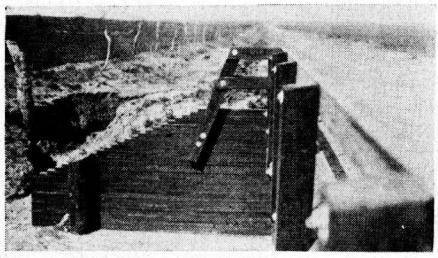

Figure 17. Wing Wall Detail in Laminated Timber. Courtesy of American Wood Preservers Assn.

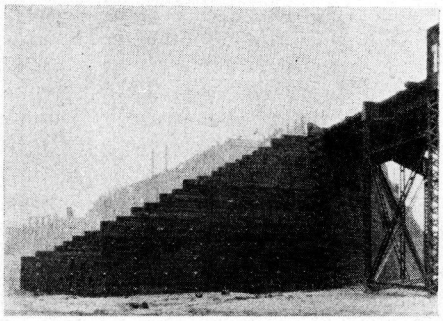

Figure 18. Interlocked Laminated Timber Bolted to Piles. Courtesy of Koppers Co.

which ribs act as the reinforcement of the finished bridge for the total loading, indicate several economies. In this way three of the disadvantages of reinforced concrete frames are eliminated, namely:

(1) Expensive falsework supports which also are an obstruction to traffic and use scarce heavy timber materials.

(2) Expensive manipulation of the heavy reinforcing, also the "jewelry" type of necessary spacers and ties for the reinforcing mats of the intrados and extrados faces.

(3) Reduction of obstructions to proper concreting placing and vibration.

Designs show that the maximum steel stresses exist during total loading so that the steel requirements are not increased by the use as a form support. Typical data for a span of 80.5 ft. are shown in a recent article in *Engineering News-Record.*[2]

Corrugated sheet metal circular arches reduce the cost of abutments by decreasing the necessary height and by providing a positive

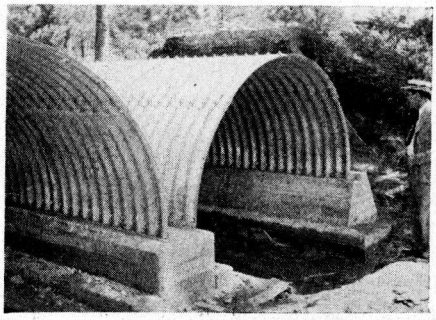

Figure 19. Shallow Gravity Wall with Arch Sides. Courtesy of Toncan Culvert Mft. Assn.

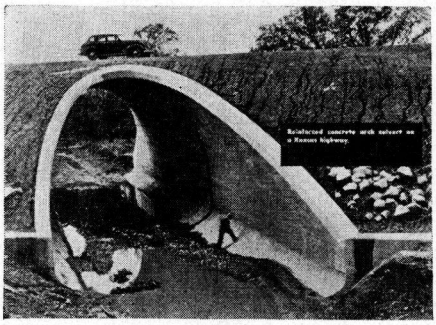

Figure 20. Reinforced Concrete Arch. Courtesy of Engineering News Record, March 9, 1944.

support at the top. Figure 19 shows the small abutment required for this type of bridge structure, and Figure 4 is a use of corrugated metal arches above high masonry abutment walls.

Elliptic and parabolic arch culverts serve the double purpose of bridge and abutments for small spans and were developed in recent years as a war measure since little reinforcing

[2] Jacob Feld, "Rigid Frame Concrete Bridge," *Engineering News Record,* May 3, 1945, p. 626.

is required. A notable example designed by the Kansas State Highway Department is shown in Figure 20.

C. TYPE OF ABUTMENTS. METHODS OF DESIGN

Considering an abutment or wing wall as a structure of two parts:

(1) Foundation, a horizontally placed unit, and
(2) Wall, a vertically placed unit, the forces acting and the resistances can be studied separately. Exceptions to this statement are such cases as sheet piling, where there is no separate foundation unit.

The wall unit is loaded by the

(1) Bridge reaction,
(2) Weights of fill resting on the back,
(3) Its own weight,
(4) Lateral earth pressure.

The wall unit must be designed to resist overstress internally, and must be connected to the foundation by such means as will overcome unbalanced tendency to translate and to rotate.

The foundation unit is loaded by the

(1) Reaction at the base of the wall,
(2) Weights of fill resting on the upper surface outside of the wall limits,
(3) Its own weight,
(4) Lateral earth pressure on the rear.

The foundation unit must be designed to resist overstress internally, and must also transmit the total resultant loads to the underlying and adjacent soil so that failure will not occur by translation (as resisted by the passive resistance of the soil on the front plus frictional resistance along the base) or by rotation (as resisted by the pressure on the soil along the base). Rotation failure can also be resisted by the uplift value of piles (or the resisting value of the imbedment detail, whichever is less), by the thrust value of batter piles or by anchorage.

The sub-soil below the foundation unit must also be investigated and design corrections made, if internal stability will not exist at any depth or along any surface, under the resultant loadings from the foundation unit.

(a) *Gravity Walls*

Gravity walls are basically masses of masonry made of such shape and size that the resultant force is within the middle third at each horizontal section. The wall may be a solid section, or to save masonry have recessed niches in the front or in the rear. The latter are respectively known as buttressed and counterforted gravity walls. (See Fig. 21 a, b, c.) The buttressed type usually becomes unsightly as refuse accumulates in the open cells. The counterforted type requires special care in backfilling and even then will show continuous fill settlement for many years. A further type is T shaped in plan, and for equal stability is the most economical in volume of masonry, especially since the stem of the T can be stepped up within the natural slope of the soil (Fig. 21 d). The stem also acts as a counterfort, reducing the earth pressure on the face wall by the frictional resistance along the vertical faces of the stem. Such frictional resistance on each face is approximately equal to the earth pressure on $\frac{1}{24}$ ft. width of wall for each foot of depth of fill. For example, a tee stem attached to a wall 20 ft. high reduces the pressure on each part of the face wall by the pressure on $\frac{20}{24}$ ft. Such reductions are resisted by the stem in addition to the pressure on the back face of the stem. A summary of experimental data and a theoretical solution is given in Appendix "A," to substantiate the recommended pressure reduction.

The abutment wall consists of the section below the bridge seats and usually a cantilever or gravity backwall to retain the fill of the road above the bridge seat level. Wing walls may be built adjoining the abutment to retain the slope of the road fills, either:

(1) in the same plane as the abutment,
(2) in a bent or curved face,
(3) in planes normal to the abutment.

When wing walls are located at each end and normal to the face wall, a U abutment is formed. The pressure on the face wall is reduced by the resistance along each wing wall, and in addition is also reduced, but by an intermediate amount, because of the arching action of the fill between the wing walls. The U form has a very high stability resistance when the clear width between wing walls is less than the height of the face wall, a condition tending to the formation of a soil arch in the fill.

Design investigation for gravity walls must check

(1) maximum compressive stresses at all sections,

(2) sufficiency of shear resistance at the junction of the wing wall to the face wall,

(3) necessity for tension resistance to prevent rotation cleavage of the wing wall from the face wall, about vertical and horizontal axes.

Such necessary resistances are often lacking in mass masonry and if not provided, the wing walls should be designed to act independently of the face wall.

(b) *Cribbing*

Whether of timber, concrete or steel units cribbing is designed as a gravity section on the assumption that all the earth fill within the limits of the framework acts with the crib units as a monolithic mass. However, all of the soil fill is not carried by the cribbing. Small scale tests (*Engineering News-Record*, Nov. 6, 1941, p. 91) as well as experience with many constructions indicate that somewhat less than 50 per cent of the fill is carried directly by the sills of the cribbing. This indicates not only a reduction of base pressures under the sills, but also that the fill aids in the resistance to sliding along the base.

If open face cribbing is considered, the type of filling used must be a heavy granular soil and have freedom from any affect from water. Non-granular materials will not arch across the openings in the cribbing and cannot be used.

Consolidation of the fill and the loss through the face will tip the cribbing forward, so that some allowance must be made for such expected loss in batter. Cribbing made up of headers and stretchers only, without a framework in the fill parallel to the face, must be so detailed that the imbedded headers are positively held and supported in position. The danger of distortion from consolidation of the fill must be avoided.

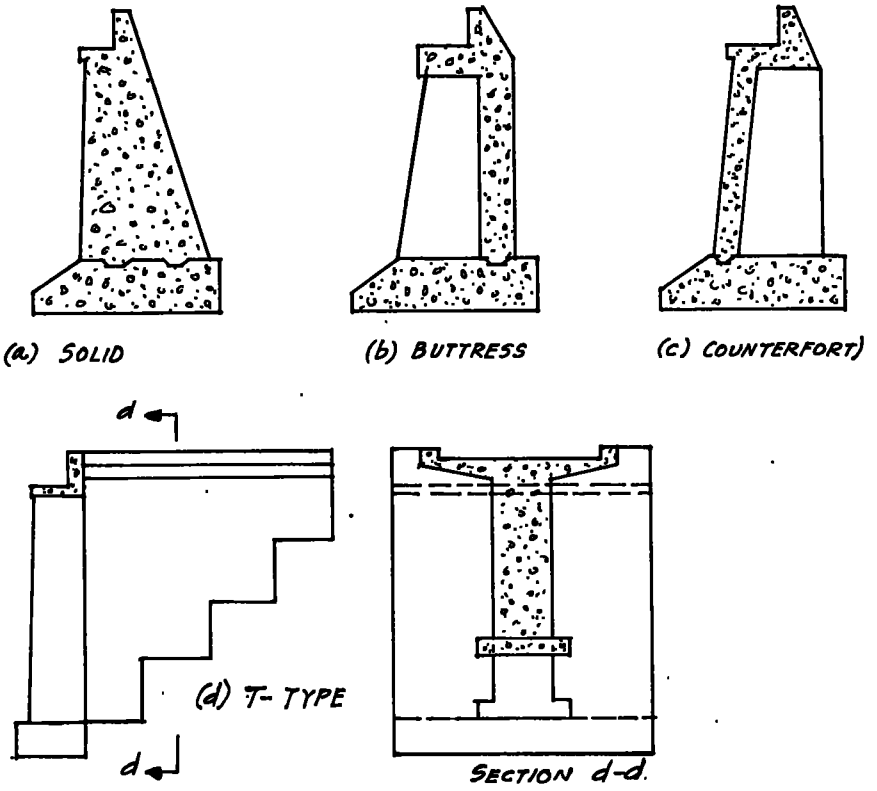

Figure 21. Gravity Types

Typical timber crib walls are shown in *Wood Preserving News* for April 1942, where basic construction as well as design recommendations will be found.

Figure 22. Rigid Frame Counterforted Abutment

ment in Figure 22, the fill can be considered as an integral part of the abutment. The value of the additional resistance so obtained must be considered in the light of the added form cost less the volume of concrete saved.

An example of counterforted abutment walls which also considered the wings as abutments is described in *Engineering News-Record* of April 8, 1937, p. 510. The author of that article, A. E. Wall, states that the choice in design was dictated by the poor bearing capacity of the sub-soil. The design is reproduced in Figure 23.

A theoretical determination of bending moments and stresses in the face slab fixed between counterforts and also along the base was developed by the Bureau of Reclamation and issued under date of July 25, 1940, under

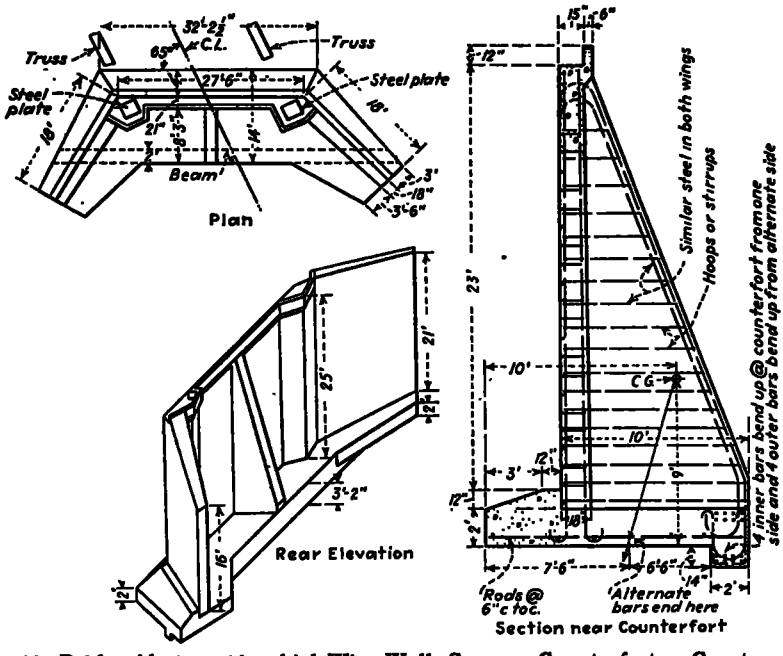

Figure 23. Bridge Abutment in which Wing Walls Serve as Counterforts. Courtesy of Engineering News Record, April 8, 1937.

(c) *Counterforted Walls*

Counterforted walls are used where the volume of necessary masonry is reduced because of economical reasons or where minimum load reactions on the foundations are necessary. If the counterforts are spaced close enough, as is shown in the rigid frame abut-

the title "Bending Moments and Shears in Counterfort Retaining Walls." The solution indicates radical departure from the stresses found from the usual assumption that the wall is a system of beams or independent strips horizontally continuous over the counterforts.

The design of an abutment as a series of horizontal beams spanning between two

counterforts placed directly below the bridge reactions was described (with graphical solution for vertical and battered walls) by W. B. Walraven in *Engineering News-Record* of Oct. 26, 1922. At that time, the design was apparently standard with the Illinois Division of Highways.

For heights below 20 ft., the design of the face wall as a reinforced concrete slab spanning vertically between the footing and a beam at the bridge seat level, which beam frames into counterforts or buttresses under the bridge reactions, has been found economical. If buttresses are used, the volume of excavation is considerably reduced. For concrete girder spans, the buttress can be designed as a beam carrying the bridge reaction as an end thrust and spanning between the footing and the bridge girder.

(*d*) *Cantilever Walls*

Cantilever walls may be used as abutments with bridge reactions carried either by individual buttresses or by the entire stem as a wall. In the opinion of the writer, in the latter case the buckling resistance of the stem must be investigated as a plate loaded along its lower edge and supported at the two (or more) bridge reactions, as axial loading and also supporting the earth pressure as a transverse loading. The stem of the cantilever wall is designed as a cantilever slab, with the additional axial loading from the bridge reactions and also the horizontal resistances developed at the sole plates. The concrete sections are under combined stress, and may be designed without regard to any live load bridge reactions.

Special attention must be given to the details in the contact zone between bridge girders and abutment seat. The fact that the abutment was designed as a cantilever wall does not guarantee the necessary conditions to permit it to act as such. If the bridge seat details prevent the freedom of movement necessary for cantilever action, the stem of the wall will become a slab supported at two ends and will develop cracks with possible failure, unless sufficient reinforcement for such action has been provided.

The combination of cantilever walls and brackets on the bottom of the bridge girders to restrict the maximum abutment movement is another case requiring special analysis. The brackets are usually so located that they are free of the abutment walls under a high temperature and will only transfer load from the abutment to the bridge girders when the abutment moves or tips. Before contact occurs, there is already a transfer of some horizontal force into the girder, limited by the frictional resistance of the girder shoe and sole plate detail. If rollers or bronze sliding plates are provided, such force can be disregarded as long as movement is not restricted. After the bracket comes into contact with the abutment, the bending stresses in the wall have changed and the bridge structure is also acting as a strut, carrying possibly 40 per cent of the total horizontal earth pressure acting on the abutment.

(*e*) *Sheeting*

Sheeting without special anchorage or buttresses can be used in several forms or combinations of shapes. A simple design, shown in Figure 10 was developed by the Division of Harbor and Bridges of the City of Toledo and consists of interlocked steel "Z" piling capped with a continuous concrete sill. The piling carries the bridge reactions and also retains the backfill. The amount of added resistance in a cantilever sheet resulting from the vertical loading is not known, and therefore had best be disregarded. Approximate methods for designing steel sheeting as cantilever are found in the several handbooks of the steel corporations.

Any formula derived for stresses of sheeting must be based on assumptions of the shape that the sheeting takes under load. In designing embedded poles subjected to unbalanced wire loading, the usual assumptions are

(1) the lateral passive resistance increases linearly with depth.
(2) the point of inflection is at $\frac{2}{3}$ of the embedded length below the surface.

These assumptions may be safely used to determine depth of sheeting embedment, for small structures, especially where reliable soil data is not available.

On the assumption that the passive resistance increases linearly with depth and the maximum value at the bottom is the same on either side of the sheeting and equals wK_pD, M. A. Drucker (*Civil Engineering*, Dec. 1934) develops a formula which approximately may be simplified to

$$D = \frac{H}{3} \cdot \frac{1 + 3K_a}{1 - K_a} \quad \text{.....(9a)}$$

Where D is the depth of embedment,
H is the height of retained fill,
K_a is the active pressure coefficient (See Table 2)[3]
K_p is the passive resistance coefficient and equals $\frac{1}{K_a}$ (See Table 2).

Formula (9a) should not be used where the coefficient of internal friction of the soil is greater than unity. When soil so hard is encountered (for values of ϕ greater than 45 degrees) the formula to be used is

$$D = H \cdot \frac{2K_a}{1 - 2K_a} \quad \text{.....(9b)}$$

self-supporting sheeting abutments is that the computed depth of embedment necessary to sustain the backfill bears no relation to the necessary penetration for the support of the vertical loading.

The use of wood piles, vertical and battered, with concrete caps is shown in Fig. 24, which is taken from the standards of the Mississippi State Highway Department. The maximum lateral resistance value of a wood vertical pile is given in A(g) of the 1943 part of this report as 8000 lb. The batter piles can probably resist higher loads, but the amount of increase is uncertain. The use of these piles for also carrying vertical loads should not affect the lateral resistance values. The bending stresses in the piles can be approximately determined by the methods employed in sheet piling design.

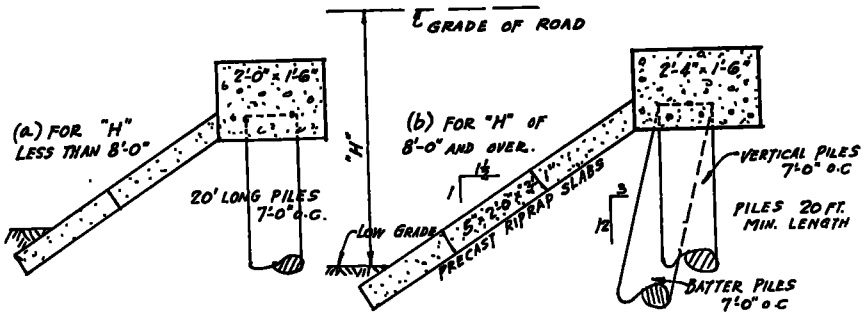

Figure 24. Mississippi State Highway Department. Capped Piles

In some designs, a steel (or timber) bent supports the bridge reactions and is placed directly against vertically driven sheeting. The sheeting beyond the span of the bent is designed as free standing cantilevers, although the portions immediately adjacent to the leg of the bent must be affected by the greater rigidity of the leg. The sheeting in back of the bent and directly below the road is supported at its top by the bridge structure. An accurate design of such a combination is impossible but a conservative use of materials is compensated by the elimination of all excavation work and the positive protection against floor waters.

A point to be carefully noted in designs of

[3] Jacob Feld, "Abutments for Small Highway Bridges," *Proceedings*, Highway Research Board, Vol. 23, p. 403 (1943).

(f) *Open Frames*

Open frames are basically braced columns or piers imbedded in the fills and supporting the bridge reactions. The type is also called a skeleton or viaduct abutment. In addition to design for carrying the vertical loads, the analysis must consider the unbalanced lateral pressure on the opposite faces of the frame. The lateral pressure in back of the frame is balanced by the lateral resistance in front of the frame. Not only must such resistance consider the age of the fill, but also the possible losses from scour, wash and erosion.

(g) *Hollow Boxes*

Hollow boxes or cellular framed abutments are box shaped masonry pockets buried in the road fill. Since the road approach is carried by the box or frame, this type is actually a

partially buried viaduct approach span also serving as an abutment. The type should only be considered where the bridge seat is over 30 ft. above the ground level. The box acts as a gravity wall and is designed as such. The sides of the box, usually braced internally by a rectangular system of struts and beams (similar to a concrete framed building) must be designed to transmit exterior earth pressure. Advantage should be taken of the fixity at edges in computing moments in the side slabs. In side elevation, the height of the box frame is reduced to conform to the contours of the ground. The reduction in dead weight and the support of the roadway approach makes this type of abutment advantageous where a soft ground pocket exists near the end of a high level bridge. Economy can best be obtained by following standard practice in concrete design and details as developed in the building construction field. Where location conditions are favorable to possible public or commercial use of the space, very little extra expense will make a revenue producing improvement from such an abutment.

(h) *Filled Boxes*

Filled boxes are "U" type gravity abutment and wing wall groups where the wing walls are tied together by structural members buried in the fill. The approach road is placed on the fill. Special effort must be expended in placing the fill to avoid overloading the buried ties and to speed up the consolidation. Walls are designed for the lateral pressure of the fill placed inside the box, considering the surcharge from the road. If the box is so narrow that a full wedge of rupture cannot form independently for each of the opposite sides, the pressures are computed from bin pressure formulas. In designing foundations for the walls, the vertical loads considered must include the weight of fill "hung up" on the wall by skin friction. An example of this type is described in *Roads and Streets, Engineering & Contracting*, Nov. 1924, p. 1038.

Walls of filled boxes can be designed by the methods developed for "Rectangular Concrete Tanks"; many examples and tables are found in the Portland Cement Association booklet, "R/C" No. 9, which is devoted to that subject.

An example of a filled steel sheetpile cell used as an abutment for a bridge span is shown in Figure 11. The sheetpiling is designed for the bursting pressure of the fill including unbalanced hydrostatic pressure during the "draw-down cycle" of tidal or flood waters. In addition, the vertical loads from the bridge and from the road must be considered.

(i) *Anchored Bulkheads*

Anchored bulkheads are sheetpiling walls which depend upon buried anchors for stability. The design of anchored or braced sheetpiling has been the subject of considerable discussion in recent years. The problem is difficult of exact solution because the acting soil pressure is a function of the deflection and the laws of the interdependence of pressure and deflection are not known. The Baumann method described in *Transactions*, A.S.C.E., vol. 100 (1935) is a graphical determination of moments and pressures from an assumed shape of the deflected sheeting. In the same reference is also described the Blum-Lohmeyer method recommended in various sheetpile handbooks, as well as a comparison with the Baumann solution.

When the anchorage consists of buried log or plate deadmen, the resistances developed can be determined from the data given in A(f) of the 1943 part of this report, under "Lateral and Pull-out Resistance of Soils." The location of the anchorage must be beyond the possible surface of fracture of the soil behind the sheeting.

When the anchorage consists of piles, either vertical or battered, with the tie rods connected near the top, the resistance of each pile is limited to the maximum values given in A(g) of the 1943 report. The stresses in the anchor piles are determined by methods similar to those used for sheeting or embedded poles under horizontal pull. Requirements of static equilibrium show that the pile must assume a reverse curve. Stresses can only be determined if the shape of the curve is assumed, or if the maximum resistance along the pile is assumed. Here again, the warning must be given, that only that portion of the pile below the lowest possible surface of fracture or rupture can be considered in evaluating the resistance.

The total resisting value of all anchorages may be less than the total lateral force acting on the sheeting, since the embedded part of

the sheeting also adds a resistance, in a manner similar to the action of cantilever sheeting. The depth of imbedment of anchored or braced sheeting need not be as great as described for cantilever sheeting. However, the depth below the low grade must be sufficient

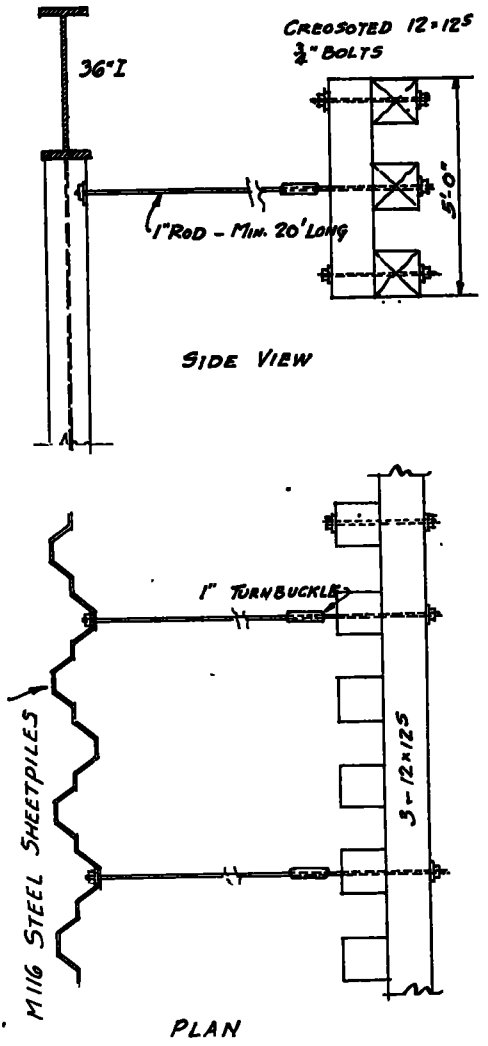

Figure 25. Anchored Piling

to prevent "blow-outs" from the maximum unbalanced pressure which would usually occur during a sudden fall in tide or flood level. The increased pressure above normal, in such a case, is described in A(a)6 of the 1943 part of this report.

Figure 25 shows a typical detail for buried anchorage, taken from the Grayling Bridge design prepared by the Michigan State Highway Dept., used in the wing walls of that bridge.

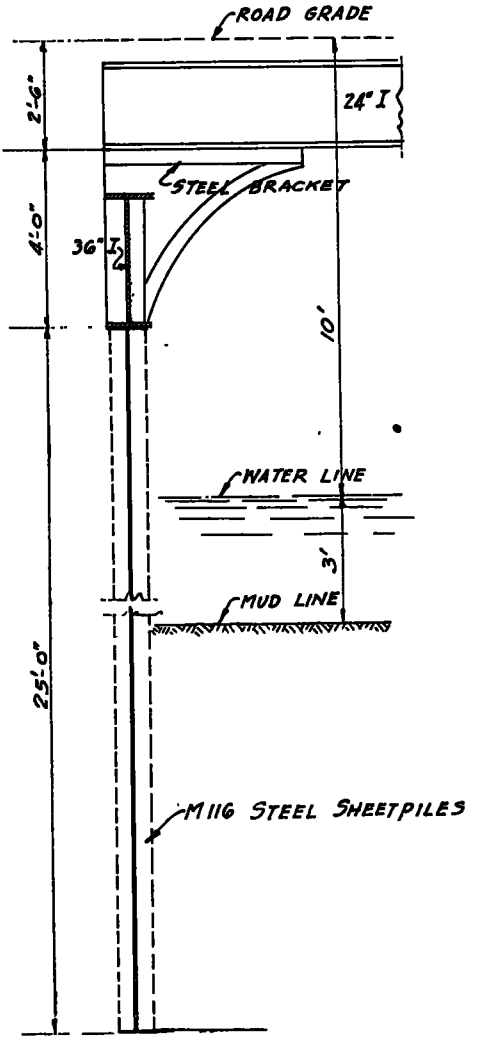

Figure 26. Braced Piling. From Michigan State Highway Dept. Design of Grayling Bridge

(j) *Braced Bulkheads*

Braced bulkheads may be either sheeting braced by the bridge structure and therefore not self supporting or sheeting made stable by vertical or by buttresses piles independent of the bridge structure. An example of the

former are the abutments of the Grayling Bridge shown in Figure 26. The top of the steel sheet piling transfers part of the earth pressure through brackets attached to the steel bridge beams into the bridge structure. Such abutments cannot be backfilled until the bridge framing has been placed and in reality the structure becomes a rigid frame with moment restraint at the girder seat. The exact design of the sheeting is somewhat complicated, even where the amount of moment restraint can be estimated.

The usual case of braced bulkheads consists of either horizontal or vertical sheeting supported laterally by the resistance of vertical piles, sometimes aided by batter piles. A typical example of batter pile bracing is shown in Figure 9. Design details of a timber structure supported by vertical piles is shown in Figures 16 and 17. Both steel and concrete sheeting can be correspondingly braced with vertical or batter piles.

The design of a braced bulkhead first consists of the design of the sheeting under assumed support conditions and checking the stiffness of the sheeting for deflections. Undue deflections affect the earth pressure, more in distribution than in total amount. The stiffer (stronger) parts of the structure will support a greater portion of the total load than is indicated from pressure diagrams drawn without regard to variable stiffness. The pressure diagram is revised so as to shift the load in direct proportion to the ability to resist, of the component parts of the structure. The reactions of the sheeting are then approximately known and the bracing piles designed for the lateral loads from the sheeting and the vertical loads from the bridge structure.

APPENDIX

Pressure Reduction at Counterfort Faces

(a) Experimental Data.

1. Winkler (1872) derives the formula

   Actual Pressure = Measured Pressure

   $$(1 + 0.116 \frac{h}{b} \tan \phi'),$$

   based on tests in a bin 15.75 in. wide and 9.85 in. high. Introduction of a center partition in the bin, increased the loss of pressure from side wall friction by 6½ per cent per wall face.
2. Darwin (1877) states that in a bin 12 in. wide, the introduction of a ½-in. partition showed a side wall friction equivalent to ½ in. width for each wall face. The bin was 14 in. high.
3. Donath (1891) reports the ratio of pressures measured in a bin 24 in. high and 24 in. wide, with and without a center partition of 0.85. This is a loss of 7½ per cent in measured pressure per wall face.
4. Mueller-Breslau (1906) in tests on a wall 5 ft. high, derives the formula

   $$-\Delta E = R \sin (\omega + \phi') \text{ as the loss per wall face}$$

   where R is the frictional resistance on each wall face
   ϕ' the angle of friction on the retaining wall
   ω the angle of the wedge of rupture.

(b) Feld in *Transactions*, A.S.C.E. 1923, p. 1580, derives the formula

$$-\Delta E = \frac{W}{6} H^3 \tan \phi' \tan^3 \tfrac{1}{2}(90 - \phi) \sin \tfrac{1}{2} (90 - \phi)$$

as the frictional loss in pressure per wall face, on the assumption of a plane surface of rupture and horizontal resistance to movement. Assuming ϕ' as fixed by the relation $\tan \phi' = \frac{3}{4} \tan \phi$, and w = 100 lb. per cu. ft.

$$-\Delta E = 12.5\, H^3 \tan \phi \tan^3 \tfrac{1}{2} (90 - \phi) \sin \tfrac{1}{2} (90 - \phi).$$

Results from this formula are:

ϕ	$-\Delta E$	$-\Delta E/E$	$-\Delta E/E$ for H =		
			10 ft.	15 ft.	20 ft.
20°	0.9 H^3	0.037 H	0.37 ft.	0.55 ft.	0.74 ft.
30°	0.7 H^3	0.042 H	0.42 ft.	0.63 ft.	0.84 ft
40°	0.45 H^3	0.042 H	0.42 ft	0.63 ft.	0.84 ft

DISCUSSION ON ABUTMENTS FOR SMALL HIGHWAY BRIDGES

PROF. GREGORY P. TSCHEBOTARIOFF, *Princeton University:* Dr. Feld's report does not go into the question of the relationship between the movement of a rigid wall and the pressure distribution against it, nor does it refer to the extensive tests performed by Terzaghi which established this relationship.[1,2] In the light of Terzaghi's research one should expect that the pressure distribution against an abutment forming part of a rigid frame bridge will be quite different from the distribution against an abutment free to move at its upper end. This and other important practical considerations, for instance depth of scour, have not been treated in Dr. Feld's report. The matter is not too complicated for presentation in a simple form. An example of such presentation is a recent paper by R. E. Fadum.[3]

DR. JACOB FELD: It seems only fair to use the latest contributions of any one author and to disregard his earlier papers dealing with the same subject. Hence, instead of referring to the papers by Dr. Terzaghi mentioned by Professor Tschebotarioff, the book by Dr. Terzaghi, "Theoretical Soil Mechanics" (published in 1943), will be quoted. On pages 84 and 85 of that book the discussion of the location of the point of application of the active earth pressure covers the following cases:

(1) When the wall yields about a fixed base, the pressure distribution is linear; for the case of vertical wall and horizontal backfill this pressure is identical with that given by the Rankine formula.
(2) When the wall yields about a fixed top, the pressure distribution is not linear, but parabolic, and the resultant acts above the third point.

The location of the point of application of the active pressure on a wall has been discussed in the 1943 report (item 10, page 405, *Proceedings*, Highway Research Board, Vol. 23. My discussion is in agreement with Dr. Terzaghi's statements as listed above. Evidently, in the case of a rigid frame abutment which is close to the case of a wall with a fixed top, the resultant pressure acts above the third point.

As to the scour, of abutments it has also been discussed in the 1943 report (page 407, item "i").

[1] "Large Retaining Wall Tests," by Karl Terzaghi. *Engineering News Record*, 1934.

[2] "General Wedge Theory of Earth Pressure," by Karl Terzaghi. A.S.C.E. *Transactions*, 1941.

[3] "Some Factors to Consider in the Design of Bridge Foundations," by Ralph E. Fadum. *Proceedings*, Thirtieth Annual Road School, Purdue University, January, 1944.