# ABUTMENTS FOR SMALL HIGHWAY BRIDGES

## PART 3

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SYNOPSIS

This report covers parts "D" (Types of Foundations), "E" (Bearing and Drainage Details), "F" (Correlation with Soil Profile Studies and Investigations) and "G" (Correlation with Previous Studies of Abutment Designs) of the general outline of all factors relating to the design of abutments for small highway bridges.

Under "*Types of Foundations*" are described the basic limitations of foundation design and the practical applications of recently developed ideas, as well as of the heritage of experience, to the proper structural dimensioning of an abutment foundation.

Under "*Bearing Details*" are described the various methods used for connecting the bridge structure to the abutment with provisions for transfer of vertical and horizontal reactions, including vibrations and temperature variations of the reactions Under "*Drainage Details*" are described the necessary precautions in construction and backfill procedures to assure that the acting loads on the abutment will closely approximate the design assumptions

Under "*Correlation with Soil Profile Studies and Investigations*" are discussed the necessary collection of physical and geological data which are used as the basis for design assumptions

Under "*Correlation with Previous Studies of Abutment Designs*" are listed the previously published reports on the standardization of abutment designs as well as several references to the standards, usually empirical in nature, set by various authorities

This report is the continuation of those published in *Proceedings*, Highway Research Board, Vol 23, page 403 (1943) and Vol 24, page 332 (1944) and deals with the sections of the general outline (Vol 23, page 403):

D. *Types of Foundations.*

E *Bearing and Drainage Details*

F. *Correlation of Soil Profile Studies and Investigations*

G. *Correlation with Previous Studies of Standard Abutment Designs*

### D. TYPES OF FOUNDATIONS

The foundation of an abutment is the supporting structural element which serves the purpose of distributing into the soil the total horizontal and vertical loads, simultaneously existing, without any local disturbance and without any internal rupture of the earth mass. Settlement cannot be eliminated when vertical loads exist, rotation or translation cannot be eliminated when horizontal loads exist The design must take into consideration the magnitude, the direction and the variations with time and with load changes of the resulting vertical and horizontal displacements

Displacements are a necessary result from the development of resistances in the soil body to support imposed loads The designer and others must realize that the settlement of a structure is not a mark of failure; no structure was ever designed or built which did not and does not change its position with changes of loading The magnitude of such movements cannot be accurately computed, but their range can be evaluated. The structural design must provide freedom of movement within that range.

Settlement, rotation and translation of a foundation result from:

1 Elastic compression of the structure.
2. Elastic deflection of the structure.
3. Compression and shear deformation of the soil immediately adjacent to the structure.
4 Consolidation of the soil body.
5. Shear deformation of the soil body.

Each of these factors causes a redistribution of reactions, and a consequent change in the calculated position of the structure. If the change in position resulting from the redistribution or change in loading is less than the original calculated movement, then ultimate stability is fairly certain, especially since the latent resistance of soils, if restrained, increases with the compression. If, however, the change in position calculated from the redistribution of reactions is greater than the movement calculated from the static equilibrium of forces, failure will certainly occur.

If a bridge is to be widened, special study must be made of the possible relative movement of both existing and added abutments. Analogous problems arise in designing additions to existing buildings, as is illustrated by the following example from the writer's practice. An extension to a multi-story industrial building was successfully accomplished by locating the floor levels of the new steel framing $\frac{5}{8}$ in. higher than the existing floor levels and providing a vertical joint between the building and its extension. After displacement, both new and old floors assumed a perfect horizontal alignment. Had the precaution not been taken, an unsightly exterior would have resulted. In bridge abutment additions, such a condition would mean an uneven drainage slope for the bridge roadway and probably cracks and failure in the road surface.

The actual load on an abutment foundation depends upon the relative rigidity of the foundation element and of the adjacent soil. A rather simple analogy is the case of a bin or silo, where the floor and wall are built on separate foundations If the wall settles more than the floor, the latter will be forced to carry more load. If the wall settles less than the floor, the load on the floor will be reduced. The same variation in loads carried can occur at the heel of an abutment foundation, where in some cases, the heel tends to deflect upwards from rotational movement of the foundation as a unit An exact analysis of this possibility, with the present state of our knowledge of soil behaviors is not possible; but an approximate study is necessary so as to evaluate rather closely the correct total load acting on the foundation

In general, if the foundation relatively "floats", that is, under a unit increase in loading it will settle less than the adjacent soil under the same unit load increase, the foundation will pick up loads from the adjacent soil until an equilibrium of the ratio, increase settlement to increase load, is reached. If the opposite state of affairs exists, then the assumed load on the foundation will be relieved, a phenomenon often called "arching", and part of the load will not affect the foundation.

Abutment foundations are usually massive or rigid structural elements and elastic deflections which cause local redistribution of reactions are not important, but loads on the abutments are seldom, if ever, symmetrically applied

### (a) *Spread Footings*

A spread footing is defined as a single continuous masonry mat, not necessarily placed on a level excavation sub-grade, on which the abutment is placed. The abutment may be of several types, such as the gravity, counterforted or cantilever walls, and the hollow or filled boxes The loads imposed consist of:

1. The vertical reaction of the abutment, from dead weight, live loads and vibration
2. The horizontal reaction of the abutment, from earth pressure, lateral bridge reactions due to traction, temperature and wind
3. The vertical loads from soil cover in front and in back of the abutment
4. The horizontal loads from the soil cover, taking into account possible water pressures from fluctuations in stream levels and ground water seepage.
5. In special locations, ice pressure and seismic disturbances
6. The dead weight of the foundation masonry

The depth of the foundation must be sufficient to protect the sub-grade from freezing, although heaving damage on abutments has not been reported as ever causing failure When placed along streams with serious flood histories, sufficient depth or positive protection against scour is essential. When placed along railroad cuts, some vibration will be transmitted into the soil under the footing; the exact effect is not determinable, but a reduction of from 10 to 25 per cent in the assumed soil bearing value is advisable, if increased settlements are not desired Such allowance has been found satisfactory in the design of foundations for buildings located adjacent to operating railroads, the full allowance applied to the foundations immediately adjacent to the trackage with reduced allowance for foundations more distant from the trackage.

The general problems of spread footings are discussed in text-books on soil mechanics, but some items especially applicable to abutments are herein considered The active earth pressure on the rear face of the foundation and the maximum passive resistance on the front face are given in Part I (*Proceedings*, H R B, Vol 23, p 403) of this report.

The limitations of accuracy in foundation design methods are expressed by L A. Palmer in a discussion of the Report of Committee on Stress Distribution in Soils, *Proceedings*, Highway Research Board, Vol 21, page 536, 1941:

> "The extent to which theory is applicable to foundation problems is mostly a matter of opinion, based on individual experience, which considering the profession as a whole, may sometimes be quite limited Furthermore, authentic data pertaining to foundation settlements are still meager
>
> "The limitations of theory in computing earth stresses caused by foundation loads can never be known until accurate and reliable means of measuring these stresses are available and are utilized on a large scale This can be the only reliable answer to the controversial question Without such evidence, doubt concerning the reliability of earth stress computations is in itself largely a matter of speculation"

The following data accumulated from experience, furnish an approach to what may be reasonably expected as the amount of settlement of a rather small uniformly loaded area

1 In soft rock, hardpan, and dense glacial deposits, settlements of from $\frac{1}{4}$ to $\frac{3}{8}$ in must be expected and will occur almost immediately under loads of 10 tons per sq. ft

2 In dry sands, a settlement of $\frac{1}{2}$ to $\frac{3}{4}$ in. must be expected and will occur almost immediately under loads from 3 to 4 tons per sq. ft

3. In wet sands, a settlement of $\frac{1}{4}$ to $\frac{1}{2}$ in. will occur immediately under loads of from 2 to 3 tons per sq ft. and will increase to $\frac{3}{4}$ in. in a period of months, especially if the sand is drained during that time

4. In dense clays, usually mixed with gravel or sand, settlements under loads of 3 to 4 tons per sq ft. will be $\frac{1}{2}$ in. immediately and will increase to about 2 in in a period of years.

5. In soft clays, settlements under 1 to 2 tons per sq. ft. will be 1 in. immediately and will increase to 3 in in 3 years There will be a continuous increase with time; values of 6 to 9 in. are not unusual.

6. In unconsolidated silts and muds, settlement amounts are indefinite.

The expected settlement under large uniformly loaded footings is rather uniform in the case of granular soils, but not in plastic clays. The total settlement of a large structure may be increased by two factors:

1 Consolidation of the subsoil by the added load.
2. Effect of adjacent loads on the footing in question

The method of computing the total vertical decrease in thickness of a layer of plastic clay or water bearing silt, based on a laboratory "consolidation test" (compression test of undisturbed soil samples laterally restrained) is now sufficiently well established. As to the effect of adjacent loads, F Kogler (Harvard Soil Mechanics Conference, 1936, Vol. 3, page 66) describes the condition where, in buildings founded on clays, the interior footings settle more than those along the street lines A related phenomenon is the upheaval of the bottom of an excavation due to the lateral pressure of the banks, causing a release of body density during the construction.

Theoretical considerations concerned with the distribution of stresses within an earth mass, caused by a point load $P$ applied at its surface, may be briefly summarized as follows. According to Boussinesq the vertical pressure at any given point within the mass equals

$$\frac{3Pz^3}{2\pi R^5},$$

where

$z$ is the depth of the given point, and
$R$ is the radius vector or the diagonal distance between the applied load and the given point.

There were rather unsuccessful attempts to correct this formula, which in reality can be applied to elastically isotropic masses only.

In the case of loads covering considerable areas, the distribution is generally assumed to be uniform as shown in Figure 1, which also gives some variations in soil reaction distribution. Figure 2(a) shows an attempt to replace Boussinesq curves by straight lines. Small scale experiments reveal a "disturbed zone" A, under the loaded area (Fig. 2b). Noteworthy is Housel's proposal (see Fig. 2c), that the vertical pressure under a load is uniformly distributed over the area formed by 1:1 slope lines from the edge of the footing. This requirement has been introduced into sumption that only the soils vertically above the footing are carried. If later study indicates that the foundation will rotate because of unequal settlement, the load carried by the heel is to be increased in amount, but not to exceed the weight of soil within the probable rupture surface. Very closely, it is bounded by a curve which starts at the heel and vertically intersects a horizontal fill surface at a distance of one-half the depth of the foundation below the surface The volume of fill

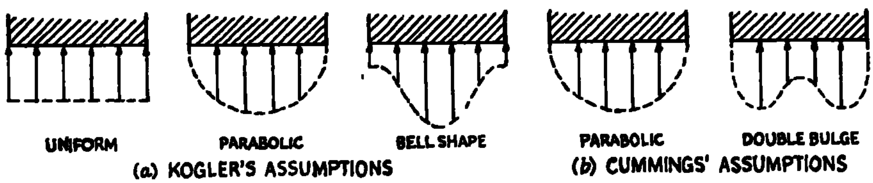


**Figure 1. Assumed Distributions of Base Loading**

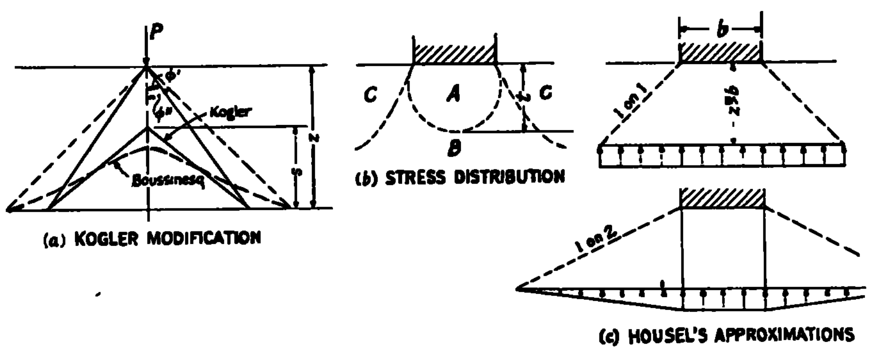


**Figure 2. Various Assumptions Regarding Subsoil Load Distribution**

the Boston Building Code Figure 2(c) shows also an alternate of Housel's proposal

To summarize the application of the above data to the design of a spread foundation supporting an abutment·

1 The reaction of the abutment on the foundation is determined from the data in Part 1 of this report, under "A". *Forces and Resistances*
2. The foundation shape and size is assumed (For approximate dimensions a choice can be based upon the data in "G", *Correlation with Previous Studies of Abutment Designs*)
3. The loads of soil cover above the foundations are first computed on the assumption that only the soils vertically above the footing are carried. [continued above] above the toe is not large enough to require any change in assumption.
4 The earth pressure on the rear of the foundation is evaluated in the same manner as for the abutment.
5. The earth resistance along the front of the foundation can be assumed to exist only if the fill is well placed and will never be removed by excavations or by scour If the foundation is poured into an excavated trench of good soil, with no forms, a large resistance will be developed to partly counter-act the overturning moment from the abutment
6 The resulting load on the sub-grade is transferred to the open soil. The

distribution is probably not linear, and certainly there is positive reaction along the full length of the base, no matter where the resultant is located with reference to the center of the base. Experience has shown that a linear distribution of pressure with the center of gravity opposite the resultant of all the vertical loads, is probably accurate enough for usual structures.

Several methods have been published to determine directly (from charts or tables) the effect of extending the toe or the heel on the base pressure distribution. Since the total load on the foundation, as well as the point of application varies with the width, no simple method can be expected. It has been found simplest to prepare two or three designs based upon reasonable assumptions and to determine the most efficient design by interpolation A theoretical solution for walls sustaining liquid pressure with triangular base reaction diagram is given by W M Borgwardt in *Engineering News Record* of June 2, 1938, page 790, but the method is too complicated to apply to the design of abutments. The method used in designing the cantilever wing walls for the Pennsylvania Turnpike, with charts for the actual design of concrete and reinforcing (under the assumptions listed) is described by William A. Jones in the *Journal*, American Concrete Institute, Vol. 15, Sept. 1943, page 5 The allowable stresses in the concrete and reinforcing of a cantilever wall foundation should follow the A.C I 318-41 Standards, American Concrete Institute, *Journal* Vol 16, June 1945, or the 1940 "Joint Committee Report" on Concrete and Reinforced Concrete. Where bond stress in the foundation governs the choice of reinforcing, liberal stress allowances are justified, since, as is pointed out by R. L. Stevens in *Civil Engineering* Vol. 9, Feb. 1939, page 125, failure in bond along an intermediate point along a bar is impossible.

### (b) *Pile Footings*

The purpose of piles is to carry vertical and horizontal loads to soil layers which can support and resist such loads when the layers are too deep for economical spread footings. The value of piles for the support of loads, vertical and horizontal, is determined by the rules given in "A" *Forces and Resistances*, (d), (e) and (g), *Proceedings*, H R B. Vol. 23, page 406-7.

For efficient location of piles careful determination of the resulting loads at the base of the abutment must be made Frame or box type abutments are often economical, where pile costs are high, since piles are then located only where bridge reactions occur (Fig. 3) Where considerable lateral forces are to be transferred into the soil and ground conditions do not permit high lateral resistances in the vertical piles, batter piles can be used at the toe line of the abutment In any case, the total load on the soil layer at the level of the bottom of piles must be analysed to prove the absence of overstress and the existence of sufficient bearing and shear resistance in the soil

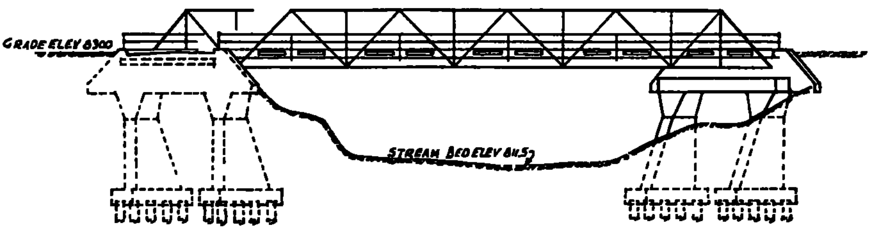


**Figure 3. Efficient Pile Foundation. From Amer. Inst. Steel Constr.: "Highway Bridge Design"**

The use of piles will not eliminate settlements In addition to the compression and consolidation of the soil layers carrying the loads transferred by the piles there is also the compression of piles themselves In friction piles, the elastic compression of the pile is computed on the basis of a reduced length, because the load is transferred by skin resistance into the soil along the length of the pile Such load transfer however increases the total consolidation of the soil since some of the load compresses layers above the level of the pile points

That the bearing value of each friction pile in a group or cluster is less than that of a single test pile, with soil and pile conditions constant, is now well established. The efficiency of a group (as a percentage of the total of individual pile values) depends upon the spacing and the number of rows An empirical reduction formula was given in Part 1, *Proceedings* H.R.B , Vol. 23, p 406. The paper by F. M Masters on "Timber Friction Pile Foundations," *Transactions* ASCE, Vol. 69, page 115, 1943, and especially the discussions of that paper have fixed some limits on the probable values of pile group efficiency A complete analysis of the Masters tests as applied to the determination of pile group efficiency, and a comparison with results from the Converse-Labarre formula cited in the Uniform Building Code of the Pacific Coast Building Officials Conference, appears in an article "The Efficiency of Piles in Groups" by J F Seiler and W D. Keeney in *Wood Preserving News*, Vol. 22, p 109, Nov. 1944 The writers also propose a compromise formula which gives values about midway between a modified Converse-Labarre formula and a pressure area load concentration criterion

TABLE 5[1]

| | Total No. of Piles | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1 | 4 | 9 | 16 | 25 | 36 | 49 | 64 | 81 |
| 1 Masters | 100 | 75 | 62 | 55 | 49 | 45 | 41 | 38 | 37 |
| 2 Converse-Labarre | 100 | 80 | 73 | 69 | 67 | 66 | 65 | 65 | 64 |
| 3 Keeney Seiler | 100 | 60 | 53 | 50 | 48 | 47 | 46 | 45 | 45 |
| 4 Empirical in Part 1 of Report | 100 | 81 | 50 | 50 | 50 | 50 | 50 | 50 | 50 |

[1] Tables 1, 2, 3, and 4 are in 1943 report.

For pile spacing of 3 ft. on centers, in square groups, Table 5 gives a comparison of *minimum* individual pile values in a group of piles

It must be pointed out that the minimum calculated pile value in a group does not represent that each pile has that value and certainly there must be a redistribution of loading to the outside piles in a group. These values do give some indication of the decrease in efficiency in pile groups as compared to individual friction pile values

An important consideration, often overlooked, is the lateral stability of pile supports. The danger of lateral displacement of an abutment, with possible tipping, exists in many bridges crossing streams or gulleys. Careful investigation of the directions and angles of slope of the subsurface layers and of the bed rock is necessary. Often, a natural or even artificial cover over steeply sloping rock surfaces, through which cover piles are driven, requires only the added impetus of pile driving or of the internal shear stress resulting from unequal pile loads introduced into the soil body, to start movement. Where such conditions exist, pile foundations are dangerous, and further study of possible alternate abutment designs should be made.

### (c) *Column Footings*

The design of individual footings for the columns of open frame abutments as well as for the base courses of cribbing requires careful analysis of allowable soil bearing values for the various conditions existing at the individual footing units. If any degree of uniformity in settlements is desired, most important are the depths of the footings, the shapes and relative sizes and the interaction of neighboring loaded footings.

The strict solution of an individual footing design for an assumed maximum unit base pressure is a function of the settlement and is many times indeterminate. The usual assumption of uniform soil resistance distribution over the entire base area is often declared to be wrong. However, there is no agreement on what the true distribution really is, even for the simplest cases. The range of disagreement runs from the claim (substantiated by some experimental data on large flexible footings on clay soils) that there is zero reaction along the footing edge to the claim of edge-action shear phenomenon (based on work by Kogler, Housel and Williams) as an important factor showing maximum base pressures along the footing edge.

As compared to a uniform distribution, the effect on the computed bending moment in a single loaded footing (whether square, round, or strip shape) may be large, but the effect on the computed shear, as well as on the unit shear and bond stresses in the concrete may be disregarded from the viewpoint of both safety and economy. In large footings, where the bending moment controls the design, as well as in combination footings of either the mat, cantilever, or combined type, the change in distribution may affect both safety and economy.

That the effect on safety is questionable can be concluded from the fact that many footings of this type have been constructed from designs based on the assumption of uniform distribution. The usual explanations as to why the concrete does not fail structurally are the surplus strength of the material (factor of safety) and the non-existent live loads included in the design. However, since unequal settlements may occur, even though the total base pressures may be less than computed, another explanation is necessary. It is found in the theory of "limit design," very similar in its fundamental idea to the theory of least work and best described by D. W. Taylor, in *Proceedings*, Highway Research Board, Vol 19, page 454, 1939. In a closed system of forces and restraints, local failure cannot occur, because the continuity of the mass causes a transfer of load to the spots that can take it best.

In conclusion several practical considerations may be stated as guides in the design and construction of individual footings:

1. The base pressure is more uniform under the more rigid footings.

2. Non-uniform distribution may be disregarded in footings resting on soil having a bearing value of over 6,000 lb per sq. ft.

3. On similar soils for the same loads per unit area, square footings settle more than rectangular footings.

4. On the same soils, for footings of the same shape, under the same average load intensity, the larger footing will settle more.

5 Footings not adjacent to other loads, will settle less than those affected by adjacent loads

6 Flexible footings may be designed for smaller bending moments than are given by the assumption of uniform distribution

7. Footing failures are seldom structural failures

8. Uniform settlements can be obtained by taking into consideration:

(a) Actual loadings to be expected.

(b) Greater soil-bearing value of exterior locations.

(c) Provision of slip joints for settlement of adjacent future structures.

(d) Effect of difference in level of footings in close proximity.

(8) Changes in portions of the soil area due to water, weathering and vibration.

(f) Difference in bearing value of differently shaped footings.

9. Maximum soil pressure under the center of a footing is no criterion of the expected settlement, because of the load distribution due to rigidity of footing

10. Bearing pressures of soil and expected settlements deduced from load tests in the field or from laboratory tests on undisturbed samples have uncertain value if the conditions of the actual construction work are not duplicated.

11. Exposure of soils during excavation may modify the soil structure and change all its characteristics.

12 All structures will settle, and provision must be made for the expected settlement by keeping them free of adjacent structures that have moved into a stable position.

### (*d*) *Processed Fills*

The use of artificially processed natural soils and fills for foundation support is rapidly developing to a technically controlled design tool Consolidation of soils by mechanical methods to any desired bearing value has been completely described in Wartime Road Problems, No 11 "Compaction of Subgrades and Embankments," Highway Research Board, August 1945. There is no reason why such methods cannot be applied to the artificial construction of foundation supports for bridge abutments located in areas of weak soils.

The use of sand columns introduced for rapid local drainage and consolidation to increase bearing value has been developed in the design of sheetpile cofferdams used in ship construction yards Chemical intrusion methods were receiving considerable attention just before the war years and should again become possible processes for altering soil characteristics. The permanence of such methods as well as colloidal control in soils must be assured before any consideration should be given to their use for permanent abutment structures

Recent investigations in the use of low

voltage-high amperage electric currents for the internal fusing of clays for sewer tunnel construction, by Robert W. Cummings, (not published), should have some important applications for foundation work in unconsolidated clay soil areas Similar experimental work in Germany is described in "Soil Mechanics" by D P Krynine, 1941, page 388

Placing of hydraulic dredged sand blankets on semi-liquid silts has been successfully used for the support of large dock and quay walls, at Spezia (Italy), Kobe (Japan) and other locations The action of the diaphragm formed in the contact zone, where the granular material and the finer grains of the original plastic mud intermix, was studied very carefully by Gen M. C. Barberis, of the Italian Navy, from 1910 until quite recently. One of the reports more easily available was presented at the 16th International Congress of Navigation, Brussels, Belgium, 1935, under the title, "Recent Examples of the Foundations of Quay Walls Resting on Poor Subsoil" This is the only report written by General Barberis in English, and makes reference to his various other more lengthy papers in Italian and French

A summary of the studies made by General Barberis as listed personally by him for the writer in 1935, is as follows.

(1) Docks built in Spezia, Italy, on a foundation of fluid subaqueous mud showed movements in accordance with the hypothesis of theories of Vierendeel and Rankine

(2) The interposition of a layer of clean sand hydraulically placed between subaqueous and rock-fill foundation of the docks shows a resistance three times as great as would be expected from the Rankine theory in the work performed from 1912 to 1915, and six times the theoretical resistance was found in the work performed from 1930 to 1934

(3) The great additional resistance resulting from the interposition of a clean, fine sand layer is due to the large cohesion between that material and the underlying mud

(4) When the sand layer is sufficiently extensive in area (say, 200 ft measured at right angles to the face of the dock) a thickness of 6 to 8 ft is sufficient to imprison the underlying clayey mud After this condition had existed 18 years, borings taken through the material indicated that the mud had not penetrated very far into the sand and that below the contact zone the mud had not been changed physically from the samples of uncovered or natural mud. Settlements of rock fill placed on the sand in 1912 had practically ceased in 1927.

(5) The artificial sand layer was dropped through 30 ft of water and spread very rapidly, and in a short time covered a large area of mud without disturbing the structure of the ancient mud layer

(6) When the sand layer is 15 ft. or more and extends for widths of 300 ft., the settlement of superimposed structure can be computed from the consolidation determined for the sand alone and without considering the effect on the underlying mud strata. Analyses indicate that a 15-ft. depth of sand eliminated any possible excessive horizontal forces in the mud layer.

(7) In the work done between 1930 and 1934, sand placed 9 ft. thick upon the mud supported loads of 2.71 kg per sq cm. or approximately 3 tons per sq. ft. This was seven times the theoretical bearing capacity of the mud layer.

(8) In the section of the quay wall at the pierhead of one of the breakwaters built in 1934, with loads of 2 5 kg per sq. cm. placed on a sand layer 15 ft. thick, there was no displacement of the foot of the quay wall, and no formation of a mud wave beyond the limits of the load. This indicated definitely that the sand layer takes the full load and distributes it to the mud without any local overstresses.

(9) In the breakwater built at Spezia, the layer of sand was 6 ft. high and supported 2 km. (1.24 miles) of breakwater Similar construction of breakwaters was used at Kobe, Japan, except that the mud was disturbed by dumping rock from the surface into a trench dredged for the sand layer.

(10) Tidal changes of the range of 6 ft. have not affected the stability of structures built on such subaqueous sand layers

As to the depth to be processed, some indication can be had from the work of Terzaghi showing that 80 per cent of the settlement under a footing comes from the soil disturbed within a depth of 1½ times the footing width, and of Cummings showing that the pressure in the soil at a depth of twice the footing width is independent of the assumed stress distribution at the base of the footing. The distribution of vertical loads through soil layers is completely covered in Chapter IV of Krynine's "Soil Mechanics" and in Chapters VIII and XVIII of Terzaghi's "Theoretical Soil Mechanics"

E. BEARING AND DRAINAGE DETAILS

The abutment is designed to sustain exterior forces resulting from the bridge and soil loadings computed on the assumption of limiting conditions. The bearings through which the bridge loads are transmitted into the abutment and the drainage facilities of the filling materials must be such that the assumed limiting conditions must be guaranteed for the entire life of the structure In these details, especial precautions must be taken to avoid changes resulting from corrosion and other natural chemical and physical phenomena.

### (a) *Abutment Seat Details*

Bridges transmit the vertical and horizontal (longitudinal and transverse) reactions into the abutment through some type of base plate or grillage. The plate may be of metal, masonry or wood, although it is usually a steel plate The bridge rests on such plate, and provision must also be made for a fairly uniform vertical base pressure under variation (1) of live load, (2) in bridge deflection and (3) in length of the bridge as affected by temperature. Provision must also be made for the transfer or elimination of longitudinal thrust resulting from temperature changes and traffic surge, and the transfer of lateral wind forces. A rocker consisting of a wide flange beam with the flanges ground to a radius, resting on a corrosive resistant steel plate which in turn rests on a phosphor bronze or similar sliding metal surface set into a recessed base plate, anchored into the masonry abutment (see Fig. 4), takes care of all possible forces. The sliding detail is only required at one end of a bridge span. In all details, provision must be made for the maximum future relative movements, in all directions, between the bridge span and the abutments, including possible wall rotation and settlement as well as bridge expansion.

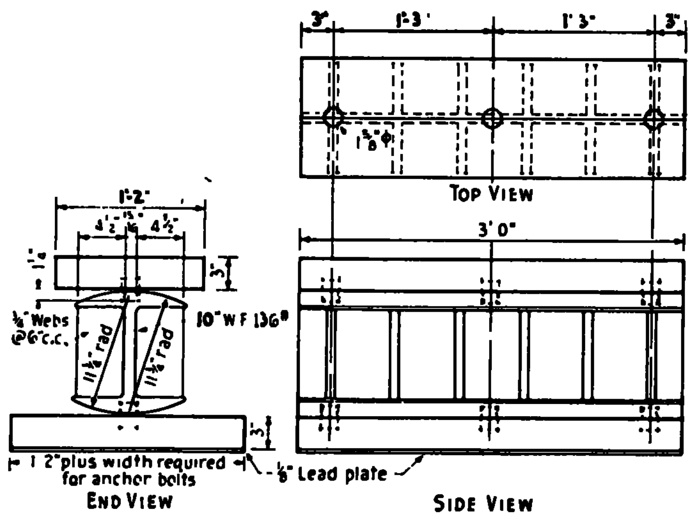

**Figure 4. Typical Rocker Detail. From Portland Cement Assn: "Concrete Bridges"**

The simplest detail of a plate resting on a plate, with or without provision for sliding, is not satisfactory because the deflection of the span under both dead and live loads causes a variation in the contact surfaces and resulting unequal bearing under the base plate. An inspection of the steel roller bearings under some 200 bridges on a railroad system (in

1925) showed that most of them had "frozen in." The standard detail was consequently changed for spans less than 50 ft. to a steel sole plate resting on three layers of heavy tar paper laid directly on a finished concrete surface, with rocker details for longer spans.

The preparation methods of the abutment seat for the base plate have undergone some change in recent years. The standard procedure for granite stone grillages was hand surfacing with a bush hammer. This method was carried over into the early concrete structures. Trowelling is usually not satisfactory because of the difficulty of getting a true concrete surface, and the floating up of fines and laitance, where maximum concrete strength is desired. Building up concrete seats above the required elevation and later tooling or grinding to proper level and elevation is preferred. The grinding tools used in surfacing concrete and terrazzo floors are available and will produce the desired result economically. The insertion of a thin sheet of lead under the base plate can be used for filling in unequal concrete surfaces, but should not be relied upon to equalize pressures under changes in the slope of deflected span

Grouting the base plate into position is the usual procedure for building columns and is sufficient when loads are fairly constant and almost always axial If grouting is used, special precautions must be enforced to make certain that the grout is mixed and aged if necessary to prevent shrinkage and grout holes are provided near the center of the base plate to release entrapped air, otherwise the plate will not be completely supported by the grout. The surface of the abutment seat under the grout should be roughened by hammer to provide a positive bond between the concrete and the grout.

Setting anchor bolts in the large concrete mass of the abutment to proper position and elevation is difficult work. Where drilling tools are available, holes can be drilled in which the anchor bolts are placed and grouted, or else over-size sleeves are set around the bolts to permit adjustment in position. The grouting should not be with fast-setting cement mortars, as is often done to save time, because the shrinkage of such mortars is quite large. The top of the grout hole or oversize sleeve should be packed with lead wool to make a water-tight seal, before the plate is set over the bolts.

An improper detail results in unsightly spalls of the concrete at the top of the abutment as well as "bumps" in the roadway at the bridge entrance. No reliance can be or should be placed upon maintenance to keep the bearing details in proper operational condition. The use of stainless steel plates and rockers, now available, will eliminate corrosion and rusting deterioration. With stainless steel, ground lenses of opposite curvature can be used for the sole and base plates to take care of deflection changes in the span. In metropolitan areas where stray electric currents must be expected, electric grounding of the bridge (if of steel) will reduce much of the corrosion and "freezing" of bridge seat details.

### (b) *Backfill Materials and Compaction*

The re-use of excavated materials for the backfill of abutments is the normal procedure. A study should be made of the cost of labor and equipment necessary to place the total backfill in the desired condition as against the cost of imported natural, granular soils.

The usual specification that backfill be "deposited in horizontal layers not to exceed 12 (sometimes 6) in. in thickness, and well compacted" is simple to write, but almost always impossible to attain. A good specification is "buildable"; a specification clause which is impossible of performance hurts the prestige of the rest of the specification. Jetting of the backfill is usually not permitted and should be prohibited as an uncertain and dangerous procedure, often resulting in semi-fluid masses which can never be completely drained and consolidated. Mechanical methods, such as the "Leaping Lena" pneumatic trench compacting plunger can be used to good advantage. Backfilling of abutments should not be left for the end of the construction program; construction traffic vibration will compact the backfill and cause soft spots to become evident. However, more walls have been tipped by sudden pressure of heavy bulldozer or truck traffic than from any other cause and such equipment must be kept far enough away from a wall or abutment to avoid influence, until the backfill is completely consolidated. Where clay soils are the only available materials for backfill a choice should be made of the types most easily compacted, with admixtures of either weathered soils or sands to permit faster drainage and consolidation.

The fill at the toe should be completed and compacted before any fill is placed in the back of the abutment, so that the necessary resistance exists to prevent movement of the structure

### (c) *Expansion Joints*

Abutments must carry the bridge as a unit so no expansion joints should be provided in the abutment structure. The wingwalls, however, have a different function, sustain a different set of loads and have different base pressures and volume changes. Wing walls should be completely separated from the abutment, by definite expansion joints permitting movement in all directions without affecting each other. Monolithic construction of abutment and wing walls almost always results in unsightly cracks which separate the portions of the wing wall which "hang up" or cantilever from the abutment and which become independent structures. If monolithic construction is desired, the wing wall must be an integral part of the abutment, connected in such manner that the wing can act as a cantilever, horizontally and vertically, of the abutment.

Expansion joints should be provided with water stops, of copper or other non-rusting materials, to continuously seal off water filtration and yet flexible enough to permit changes in alignment across the joint. Keyed joints are not advisable since the smaller concrete thickness at the key forms a weak section which is often spalled by the pressure either of frozen moisture in the joint or of expansion arching. The resistance provided by a key can be more safely obtained by steel rod dowels, one end encased in sheet metal sleeves. Dowels of ¾-in size can deform without damage to the masonry if the adjacent wall sections tend to change position relatively.

### (d) *Stabilization of Backfill for Road Base*

The road surface approaching a bridge must be at proper grade to avoid the sudden bump which not only affects traffic flow but also does no good to the abutment and bridge structure. If the necessary stabilization of the road base cannot be provided, an approach slab, about 20 ft. long, should be designed to span the gap between the abutment wall and undisturbed ground. The use of temporary roadway surfaces at bridge approaches so often used in practice, should be discouraged Information concerning methods and tools which can consolidate soil fills to any desired density, covering all types of soils, may be found in the Highway Research Board, Wartime Road Problems, No. 11, issued in 1945.

### (e) *Drainage Methods and Details*

Where no positive drainage method is provided for the backfill, the abutment should be designed for a pressure not less than that of water The materials used for drainage layers must be such that they will not lose porosity. For instance, a layer of sand placed against a wall which is backfilled with clay will soon clog up completely, at the same time providing space for the clay to expand, take up water and later (in a dry spell) shrink and crack. The next storm will fill the cracks with water and hydrostatic pressure acts on the wall. This condition was found to exist and was the cause of failure of several concrete basement walls in Hartford, Connecticut. Similar failures were noted in Detroit, Michigan, also in clay soil, and from the use of silt backfill which liquified in spite of a drainage layer of gravel.

The standard detail for railroad bridge abutments about 1915, was to call for hand laid dry stone for the entire length and for the full height of the abutment. Since the walls were usually built with stepped backs, the rubble was partly supported at each step. The high labor cost of the stone-work caused a change in the method and gravel layers, often in porous cloth bags, about 12 in. thick are now the standard detail for both railroad and highway bridge abutments. Placing gravel against a high wall and keeping it in position during proper backfilling operations, is not a simple task. A simpler and more positive drainage layer, and not more costly if the gravel is carefully installed, consists of dry laid porous concrete blocks with hollow cells laid vertically, using a thickness of 6 or 8 inches. The top course should be laid with cells horizontal to prevent sealing the open channel.

No drainage layer has any value if provision is not made for the free exit of accumulated water. A continuous pipe drain, not less than 6-in. size, is laid in the bottom of the drainage layer, usually just above the level of the backfill on the front face of the abutment. The drain should have a slope of not less than 2 per cent, with tile or concrete weep-holes through the wall. Where road drainage is provided, as in grade separation structures, the weep holes are continued into pipe connections to the nearest basin or manhole. Where frost conditions are prevalent, all pipe outlets and connections should be below the frost line. Otherwise, the drainage system will be of no value in the early Spring, with ground still frozen and the usual heavy rains.

The bridge seat should slope downward at least on a 1 per cent slope and the edge of the seat should be provided with a projecting coping beyond the face of the abutment to act as a drip. If the face of the abutment is ornamented, with stone or otherwise, the drainage of the bridge seat should be concentrated by formed channels and collected into a drain pipe or leader, which can be buried into the abutment Such drain should be capped with a roof leader inlet equipped with a screen and basket to prevent clogging.

### F. CORRELATION OF SOIL PROFILE STUDIES AND INVESTIGATIONS

In 1914, an American poet, Robert Frost, in "North of Boston" expressed simply and completely the necessity for soil profile studies and investigations·

"Something there is that doesn't love a wall,
That sends the frozen-ground-swell under it,
And spills the upper boulders in the sun,
And makes gaps even two can pass abreast

Before I'd build a wall I'd ask to know
What I was walling in or walling out,
And to whom I was like to give offense
Something there is that doesn't love a wall,
That wants it down "

The minimum information which should be collected will answer:

1. What types of soil and at what depths will they be excavated?
2. Where is the ground water level normally and how high may it go?
3. What is the nature of the soil on which the foundation will rest and what uniformity do local conditions indicate in the soil layers when cut horizontally?
4. What types of soil underly the foundation level, at least to a depth of 10 ft. below such level?
5. Are there any signs of local slips in the soil body and is the bed-rock below the soils fairly level?
6. Is the local material easily compacted as backfill, or can other material be economically brought to the site (with the cost of disposing the excavated material taken into consideration)?

The various types of boring methods and test pits and a description of field tests to determine soil bearing value are found in Chapter X of Krynine's "Soil Mechanics." An inspection of structures in the vicinity of the proposed bridge abutment, with a critical eye and an open mind, will often disclose pit-falls to avoid A test pit may only show the type of soil at the point where it is dug, but it does show the entire picture of what will be encountered and is worth the added cost over that of borings

In the technical literature of the past 20 years, there are very few reports of abutment failures, not resulting from scour under the foundations by flood waters. The only other cause reported seems to be internal soil body failure, either from shear slipping or from volume loss due to consolidation and bleeding of soil moisture. In every instance of failure, a little more preliminary investigation would have prevented the unpleasant and costly occurrence. Failure to collect sufficient data can be obviated if the advice of E. W. Wendell of the N. Y. State Dept. of Public Work (*Proceedings*, A S C.E. Oct. 1940, p 1563) is taken: "The organization that conducts the field activity covering sub-surface investigations should be a part of the construction organization."

### G. CORRELATION OF REPORT WITH PREVIOUS STUDIES OF STANDARD ABUTMENT DESIGNS

There are very few published studies of standard abutment designs In the early authoritative engineering works, Baker's "Treatise on Masonry Construction," first issued in 1889, gives a complete description of gravity, U and T abutments with tables of dimensions and quantities for walls from 5 to 35 ft. in height. This data seems to be the basis of all subsequent treatments of the subject, as far as masonry and mass concrete abutments are concerned.

Reinforced concrete cantilever and counterforted retaining walls are discussed by Taylor and Thompson (1905) in "Concrete, Plain and Reinforced," but no examples of abutments are mentioned. The literature on retaining walls is quite copious and standard wall designs for military purposes were developed (probably empirically) by Vauban in the early part of the 18th century Reference to many of the original contributions to retaining wall design have been published by the writer elsewhere.

(Jacob Feld: "History of the Development of Lateral Earth Pressure Theories," *Proceedings*, Brooklyn Engineers' Club, Jan 1928, p 61–104
Jacob Feld "Review of Pioneer Work in Earth Pressure Determination" *Proceedings*, Highway Research Board, Vol 20, p 730, 1940)

In the June 1940 *Proceedings* of the American Society of Civil Engineers, there appeared the report of the Special Committee of the Structural Division on Masonry and Reinforced Concrete on "The Practice of State Highway Departments in the Design of Abutments" The report is a compilation of answers received to a questionnaire issued in 1938 to all the State highway Departments, from all but two of the States Special attention is directed to a necessity for more careful analysis of the design, especially of the foundations Large variation in the requirements covering type of backfill materials and procedure of placing the backfill is found, even in adjacent States Quite opposite expressions are noted in the types of abutments used, some States being definite that skeleton type or frame abutments are not economical and others reporting almost exclusive use of this type because of its economy It is quite evident, that the impact of habit and the weight of procedure overshadows independent study and design, in possibly more than half of the State highway offices

Some time ago, the writer submitted a design for a two span continuous girder bridge with reinforced concrete vertical slab abutments, using pilasters set in shale rock as supports for the girders and as beams to carry earth pressure The design was rejected by the State highway department to which it had been submitted for two causes:

1. The county was to pay for the bridge and the State contracted to pay for the road surface and this design did not permit a physical separation of the two obligations
2. The State standards permitted gravity abutments only.

A redesign of concrete arches against gravity abutments, covered with sand fill on which a concrete road surface was later built, all at a cost of over twice the original design, was finally accepted This example shows that further reliance on present obsolete standards and precedence seems now out of order, and full advantage should be taken of the latest contributions to the proper and economical design and construction procedure of highway bridge abutments

ADDENDUM TO PREVIOUS PARTS OF THE REPORT

A (a) The application of earth pressure formulas to abutments and wing walls traversing a sloping bank of earth at an oblique angle and with the backfill having a broken slope is given by C H Lang in *Engineering News-Record*, Feb 22, 1945, page 90

A (f) The resisting capacity of anchors or deadmen is reported by V A Eberly in *Engineering News-Record*, Dec 2, 1943, page 833, with a discussion by A Woolf, Mar 9, 1944, page 125 The data given is consistent with the values recommended in this report

A (i). The seriousness of flood damage to bridges is again discussed in the Progress Report of the Chairman, Joint Division Committee on Floods, *Proceedings*, American Society of Civil Engineers, Feb 1944, page 170, under the heading "Highway Damage Leads,"

"In occasional instances, bridges which had been rebuilt were damaged again by later floods, often due to undermining of abutments or piers In this connection, a vast amount of experimental work in the past has been aimed at discovering the effect of bridge-pier shapes on stream flow, but apparently little has been done to determine the extent to which various pier shapes affect scour of the river bed This is a field worthy of investigation, in which careful field determinations are needed in conjunction with model studies Of interest is the rule of thumb, derived from hard experience, used by railroad engineers in the Great Plains region of the United States, where river beds are notoriously sandy and subject to deep scour during floods. This so-called 4 to 1 rule requires that bridge foundations be placed to allow for a bed scour of 4 ft. in depth for each foot of rise in water surface above the low-water plane"

C (c). A novel example of counterforted wall built in India is described in *Engineering & Contracting*, June 1926, page 283. Counterforts each consist of four concrete struts within a triangle between a continuous concrete buried heel footing and vertical columns in the face of the wall, spaced about 20 ft apart for a wall height of 20 ft. The wall panels between columns are concrete block barrelled arches, with a continuous footing for the columns and the wall panels. The design seems easily applicable to use as an abutment, with possible use of pre-cast concrete columns and struts

C (h). A variation from the usual type of cellular wall is described in *Engineering News Record*, May 13, 1926, page 768, used in St. Paul. A continuous vertical wall 16 to 27 ft. high is built with ribs projecting into the fill 13 ft. to a continuous vertical tie wall, 6 ft. high and buried 9 ft. below the surface. No footings are provided for the ribs or for the tie wall. The type was chosen because it introduces the least toe pressures on the footings of all retaining wall types

D (d) After completion of this report, the writer was in correspondence with Mr. Robert A Cummings who was kind enough to communicate the following details concerning his method of clay stabilization by the use of electric currents

1. The occurrence, the chemical and the physical composition of each clay must be known
2. Nearly all clays are reversible colloidal materials, and subject to solidification by controlled heating.
3. It is quite unnecessary to carry the heating to an incipient fusion
4. The proper temperature must be determined experimentally as it will vary with the clays.
5. I have observed that variations from 800 deg to 1200 deg have occurred. In the absence of more precise reading a cherry-red color was sufficient to eliminate combined water. Of all soils clay seems to have the largest water carrying capacity. The water extraction problem is difficult to deal with because of the cost. I have not tried the vacuum extractor of water which has been successfully used by reinforced concrete contractors It indicates a simpler solution. Then the well-point can be used for very wet soils. In any case "the punishment must fit the crime".
6. The electric current used for heating was an AC transformed to low voltage and high amperage If you will take an ordinary wire grid of 2-in mesh and heat it electrically, you will gain a much better concept of the operation, especially if the wires are covered with a brick clay coating, ½-in thick.
7. The physical aspect of marl is an example of the chemical composition that influences physical characteristics