tests are given by Professor Tschebotarioff in his paper printed in the January-1948 Proceedings of Am. Soc. C.E. According to these the effectiveness of a vertical sand dike is: 100% when t = 1.0 i.e. when the dike thickness is equal to its height; 50% when t = 0.5; zero, when t = 0.1. The effectiveness indicated by the analytical method as shown on Fig. 3 is 100% for t = 0.7; 77% for t = 0.5 and 17% for t = 0.1. Since there are no test data for values of t between 1.0 and 0.5 there is no actual check on the minimum thickness of a vertical dike required for 100% effectiveness.

According to the tests the full effectiveness of a trapezoidal sand dike is attained when the dike is placed on the natural slope (1: 1.73) with a maximum top berm of 0.2H while analytically the full effectiveness should be obtained only when a top berm is o.4H. This discrepancy may be due to the probable error in measurement readings, estimated to be 5% . Errors of this magnitude are sufficiently large to account for the difference. Thus the total pressure for the trapezoidal dike with berm width t = 0.2 is $10.5H^2$, whereas for the level sand bank it is $10.0H^2$, a 5% difference. Another possible explanation is that the fluid clay in the lower regions may have partially consolidated and did not exert full liquid pressure against the dike.

The present tests do not as yet furnish enough data to determine the variation of the effectiveness of the various dike types with their dimensions and the nature of the dike material; neither did these tests determine the influence of a surcharge on the effectiveness of the dike. The tests are still in progress; it is expected that the future tests will supply the missing data for a full comparison with the analytical method.

CONCLUSIONS

1) Active pressures against quaywalls from backfills and superimposed surcharge may be reduced as much as 70% by means of sand dikes.
2) The sand dike effectiveness increases with the increase in ratio of the bouyed weight of dike material to that of fluid backfill.
3) The rectangular dike shape uses the least borrowed material but is expensive to place and should be used only in special cases.
4) The triangular dike ranks second in material economy, is easily placed and should be used wherever possible.

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A STUDY OF RETAINING WALL FAILURES

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SUMMARY

Under the auspices of the American Railway Engineering Association, questionnaires were sent to the chief engineers of all the American Railways requesting information about retaining walls or abutments that had failed or that had experienced progressive movement to an undesirable extent.

The questionnaire brought forth information about a large number of retaining walls and abutments. The failures and movements have been analyzed with respect to their probable causes. It has been found that, with very few exceptions, the difficulties were due to misjudgment of foundation conditions rather than to incorrect assumptions regarding the backfill pressure.

INTRODUCTION

In 1945, the Committee on Masonry of the American Railway Engineering Association appointed a subcommittee on earth pressures against masonry structures. The principal assignment of this subcommittee is to study and revise the current specifications of the Association with respect to retaining walls and abutments.

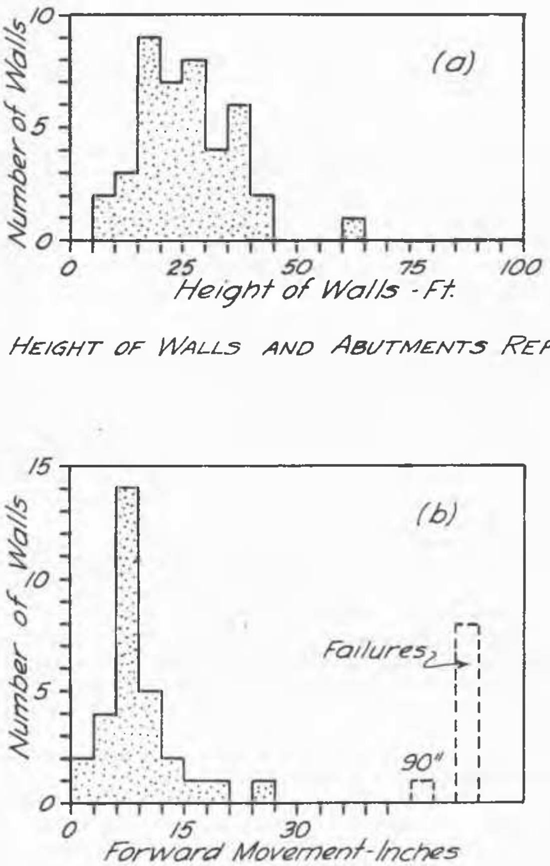
As one of the initial steps in this study, a questionnaire was sent to all of the principal railroads of the United States and Canada to obtain information about retaining walls and abutments that had performed in an unsatisfactory manner. In particular, information was requested concerning retaining walls that had failed completely or that had experienced movements of such magnitude that their function had been impaired. Data were not requested, however, about failure of abutments by scour.

The chief engineers of 77 railroads were invited to contribute information. Of this number, 37 or 48 % did not reply. An additional 24 or 31% reported that no difficulties of any consequence had come to their attention or that failures had occurred only in very old walls designed according to rules of thumb now considered obsolete. The remaining 16 or 21 % reported that the behaviour of at least some walls and abutments had been unsatisfactory enough to cause concern. These 16 submitted information about 44 walls and abutments that were considered unsuccessful. The location of these structures is shown in Figure 1.

Inasmuch as almost 80% of the individuals to whom questionnaires were sent either reported no difficulties or did not reply, it would appear to be a reasonable conclusion that the great majority of retaining walls and abutments can be considered successful and that as a rule the present methods of design

LOCATIONS OF EXAMPLES REPORTED

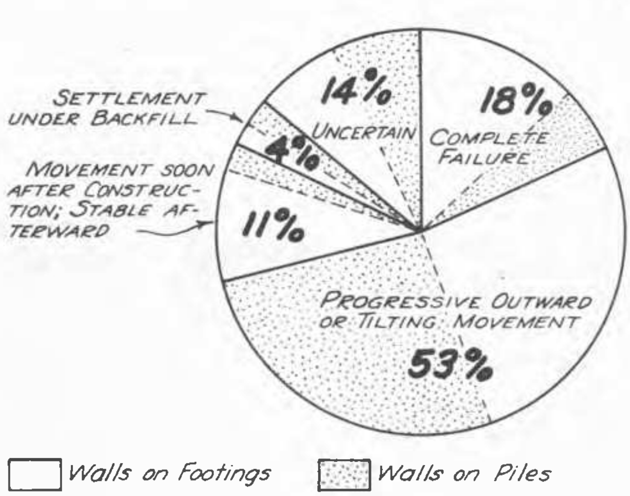
FIG.1



HEIGHT OF WALLS AND ABUTMENTS REPORTED

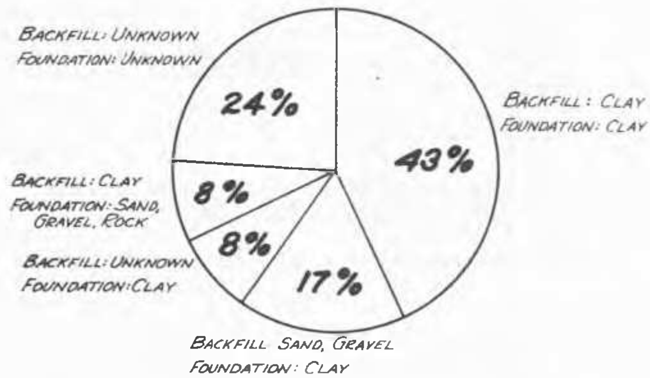
REPORTED MOVEMENT OF WALLS AND ABUTMENTS

FIG.2



TYPES OF UNSATISFACTORY BEHAVIOR

FIG.3



BACKFILL AND FOUNDATION MATERIALS
WALLS SUBJECT TO PROGRESSIVE OUTWARD MOVEMENT

FIG.4

are at least adequate and possibly conservative. Nevertheless, a sufficient number of failures or examples of excessive movement were reported to indicate that walls designed according to the customary procedures are not necessarily stable or static.

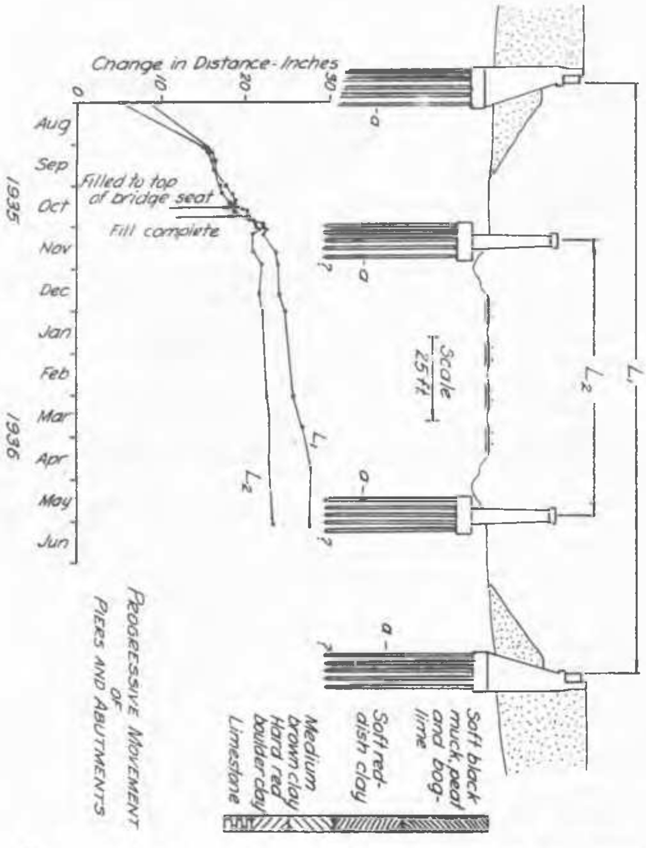
DESCRIPTION OF WALLS AND MOVEMENTS

The unsatisfactory walls and abutments are classified in Figure 2a according to their height. In Figure 2b they are classified in accordance with the magnitude of the movement they experienced. The majority experienced forward movements of 6 to 12 inches. This is probably an indication that movements less than 6 inches are commonly not a matter of concern and it suggest that movements smaller than about 3 inches are usually considered quite normal and satisfactory.

In Figure 3, the walls are classified in accordance with the type of movement that they experienced. It may be observed that about 18% failed completely. That is, they either overturned, broke structurally, or were considered in such imminent danger of collapse that they had to be strengthened or removed and replaced. Over half of the walls experienced a progressive outward or tilting movement.

Figure 3 also indicates that almost half of the unsatisfactory walls were supported on piles. This fact suggests that there was general recognition of unsatisfactory foundation conditions and that an attempt to improve the foundations was made by providing pile support. Nevertheless, it does not appear that the mere use of piles, even including batter piles, was sufficient to prevent excessive movement of the walls.

Figure 4 shows the foundation and backfill materials associated with those walls that experienced progressive outward or tilting movements. It may be observed that in every reported example of progressive movement where there



Progressive movement of piers and abutments

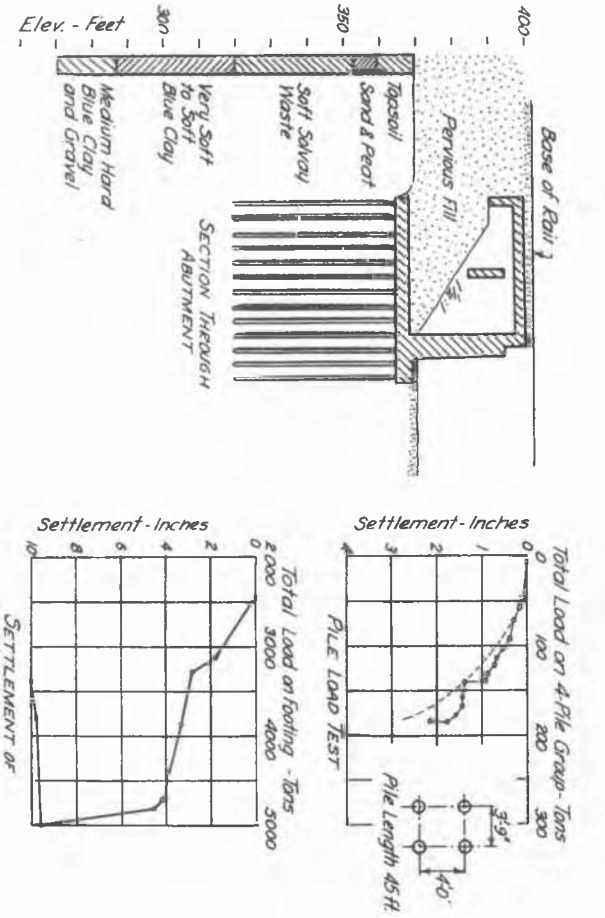
FIG.6

was sufficient information to classify the soils, clay occurred in the foundation or in the backfill, and in most instances occurred in both. This suggests that present design procedures and those that have been used in the past must be quite conservative when foundation conditions are good and when the backfill consists of sand or gravel. On the other hand, it also suggests that the factor of safety of walls founded on clay or backfilled with clay is probably on the average much lower than believed by the designers.

TYPICAL EXAMPLES OF UNSATISFACTORY BEHAVIOR

The first example is illustrated in Figure 5 which shows the pertinent data concerning one of a pair of open abutments constructed for a grade separation. It was recognized that the foundation for the abutments would require pile support and the structure was founded upon cast-in-place concrete piles having an embedment of 45 ft. One group of four of these piles was tested with the results shown in the figure. On the basis of the test, it appears that the average shearing resistance of the soil was approximately 0.17 ton per sq ft.

To lighten the load on the abutments as much as possible and at the same time to reduce the active earth pressure of the railroad embankment, the abutments were made hollow and it was specified that the fill should slope downward toward the toe. In spite of these precautions. each abutment began to settle during construction and when the total dead load reached 5000 tons each abutment began to tilt toward the fill and to sink into the ground. At this stage the average settlement increased very rapidly from 4 inches to almost 10



Failure of abutment due to exceeding bearing capacity of subsoil

FIG.5

inches, whereupon the fill was removed and the design altered in such a way that no fill rested upon the bases of the abutments when the bridge was completed.

On the assumption that the average shearing resistance of the soft subsoil was 0.17 ton per sq ft, the total ultimate bearing capacity of the base of the abutment should have been approximately 5300 tons. This value is in reasonable agreement with the observed load at failure. Hence, it appears quite obvious that the failure of the abutment was caused by overloading the subsoil and that it had little if any relation to the active earth pressure against the abutment.

Figure 6 shows the pertinent data concerning a typical example of progressive forward movement of large magnitude. The piles may have penetrated the soft material and rested on the medium clay at a depth of 40 ft, but the records indicate that more likely they terminated near the points marked a, within the soft clay deposit.

The two abutments and two intermediate piers for the bridge were completed and backfilling was in progress when movements of all four elements of the substructure were observed. Measurements were made with the results indicated on Figure 6. It may be noted that the distance L_2 between the intermediate piers decreased almost as much as the distance L_1 between abutments. This means that the movement was deep-seated and involved the lateral squeeze or flow of clay toward the center of the bridge from each end. Such a movement could hardly be caused other than by overloading the clay stratum by the weight of the backfill.

Under the weight of the fill, the bearing capacity of the soft reddish clay and overlying material was probably approached and a slow lateral flow or creep was initiated by the excessive shearing forces. The lateral pressure against the abutment was probably not excessive because the abutments were of the open type and the fill was allowed to extend through them.

This example demonstrates that the lateral forces acting against a vertical section through an abutment supporting a high fill may be very great, considerably greater than the forces due to the active earth pressure against the abutment itself. This fact was demonstrated clearly by the behavior of a structure built during the war to store iron ore. The structure consisted of two parallel retaining walls about 30 ft high and 275 ft apart. The walls were tied to each other by a series of steel rods located just below the ground surface. A deep deposit of medium clay was located beneath the structure. The behavior of the walls was observed by means of strain observations on the tie rods. At the yield point of the rods, their capacity to resist the horizontal forces against the retaining walls was 70,000 pounds per lineal foot of wall. When the iron ore was piled to a height of 22 ft, the rods reached their yield point strain. Yet, according to any rational method of computation, the active earth pressure of the ore against the walls could not have exceeded about 16,000 pounds per lineal foot. Therefore, the actual horizontal forces exerted against the walls and their foundations were over four times the computed earth pressures. Independent observations demonstrated that the ultimate bearing capacity of the clay beneath the storage yard was 2.6 tons per sq ft whereas the weight of 22 ft of ore was 1.8 tons per sq ft. Therefore, the factor of safety against a bearing capacity failure was only about 1.4. At such a low factor of safety, excessive and continuous horizontal deformations in the subsoil are to be expected.

In a large number of the other examples of progressive outward movement or tilting, conditions appeared to be similar to those indicated in Figure 6 except that the movements were generally much smaller. This would suggest that the shearing stresses in the foundation were considerably smaller with respect to the shearing strength of the soil than in the example described. Nevertheless, in practically all of the examples it is believed that the lateral forces in the subsoil of the structure were considerably greater than the computed active earth pressure. Progressive movements were even observed on several walls supported by fairly stiff clays.

In a number of instances, progressive movements seemed to begin in the 1930's in spite of the fact that the walls had been apparently static for many years before. This was generally attributed by the railroad engineers to the marked increase in locomotive weights during this period. Many of the walls were designed for Cooper's E-30 or E-40 loading, whereas by the 1930's the weight of motive power had generally increased the loading to E-60 or E-70. It is quite possible that this increase in live load, with accompanying increase in toe pressures, was responsible for the initiation or revival of movements. However, there is no satisfactory way to evalute the relative importance of this factor.

In most instances where both the backfill and foundation consisted of clay, it was not possible to ascertain the relative importance of those movements associated with overloading the clay foundation and those that may have been caused by the progressive decrease of the strength of the backfill material. The evidence appears to indicate that foundation failures or foundation movements were more prevalent than those caused by an increase in the backfill pressure of clay. However, several of the complete failures undoubtedly belong in the latter category and it is certain that clay backfills in several instances did exert an increasing pressure.

In two examples concrete gravity walls were built to retain clay fills sloping at $1\frac{1}{2}$ to 1 upward and away from the crest of the wall. In one case, the top of the fill was about 8 ft above the top of the wall and the wall itself was 8 ft high. In the other, the wall was only 6 ft high and the fill was about 60 ft high. Both walls were stable for a number of years. Both walls failed structurally at a point above the foundations. This fact indicates that deep-seated foundation movements could not have been the primary cause of failure. Hence, it must be inferred that the pressure of the clay gradually increased until failure occurred.

CONCLUSIONS

The records obtained by means of the questionnaire are still being studied and analysed. However, it is believed that several rather definite conclusions can be drawn at the present time.

1) Considering the total number of conventionally designed retaining walls and abutments, failure or progressive movement is a relatively uncommon occurrence.
2) Unsatisfactory behavior is rarely encountered unless the subsoil of the wall, or its backfill, or both, contains clays or clay-like materials. The most prevalent cause of trouble is the overloading of clay foundations by the weight of the backfill. The second most common cause is probably the gradual increase of pressure when the backfill material consist of clay. A possible third cause of some importance is the increase of live load.
3) Contrary to the opinion of some engineers, it seems doubtful that the effects of vibration due to traffic on granular backfills are of serious importance. Otherwise it is probable that more difficulties would have been reported with walls backfilled with sand.
4) The unsuccessful behavior of walls has been due primarily to misjudment of the foundation conditions. Present methods of design place the emphasis almost exclusively on computed earth pressures that often have little relation to the real forces that the structures must resist. In the future, much more attention should be given to the foundation conditions and to possible time-conditioned changes that may occur in the backfill. The theoretical study of earth pressures has provided a fascinating diversion for engineers, but it has tended to blind them toward an understanding of the real behavior of earth-retaining structures.

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