CHAPTER 16

SUBSTRUCTURE DESIGN OF THE NEW MYSTIC PIER NO. 1, BOSTON

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In order to provide adequate modern terminal facilities to handle the anticipated commerce requirements, the Port of Boston Authority, in 1947, prepared a master plan for the future development of the port.* The second step of this plan was the reconstruction, on modern standards, of the old Mystic Pier No. 1. Planning for this work was started in 1949 and the new pier was opened to commerce in September 1952.

The Mystic Pier is located in the Charlestown section of Boston, adjacent to the freight yards of the Boston and Maine Railroad Company, and extends out into the Mystic River. Its position in relation to the City of Boston and other port developments is shown in Fig. 1.

The new pier is shown in Fig. 2. It has an over-all width of 468 ft, with about 600 ft of berthing space along the northern edge and about 900 ft along the southern edge. The greater part of the pier is covered by a transit shed, 418 ft in width and 538 ft long. The shed has a steel framework, the walls consist of concrete extending up to 6 ft above the floor with transite siding above, and the roof is of precast concrete plank with a generous allotment of skylights. Office space is provided at the west end of the shed and on the landward side of the shed are two covered loading platforms, each 200 ft in length, and a battery charging building.

Outside the transit shed there is a working apron 20 ft wide at the east end of the pier and 25 ft wide on both the north and south sides. Five railroad tracks extend practically the full length of the pier, one along the northern apron, one along the southern apron and three are located in a depressed trackwell along the center of the transit shed.

The general locations of these facilities are shown on the site plan in Fig. 3.

Site and Subsoil Conditions

The new Mystic Pier was constructed at the site of the old pier which had existed, in various forms, at this location for about 85 years. This old pier, which is shown in Fig. 4, consisted essentially

*See Chapter 22 by G. L. Wey

of a fill which was held in place by masonry retaining walls supported by timber piles, and surrounded on three sides by a wooden platform supported by timber pile bents. The greater part of the pier area was covered by two timber freight houses which were founded on spread footings resting on the fill. It was estimated that the dead load plus live load storage capacity of the freight houses was equivalent to a maximum area load of 430 lb/sq ft. The bottom of the channel around the pier was approximately at El. -35.0 at the east end of the pier and at El. -30.0 along the north and south sides.

The subsoil conditions underlying the pier were investigated by twenty-five exploratory borings and two undisturbed sample borings. These borings showed that the soil strata underlying the pier are relatively consistent. The elevation of the ground surface of the old pier was approximately El. 13.5. Beneath the upper layer of miscellaneous fill are strata of fine silty sand and coarse sandy gravel, the lower boundry of the sandy gravel varying between El. -21.0 and El. -26.0. Below the sandy gravel is a deep layer of soft blue clay, which, over most of the area, has a thickness of about 85 to 90 ft. The clay is underlain by compact sand and gravel, hardpan and bedrock.

A typical section through the east end of the old pier showing the subsoil conditions is presented in Fig. 5.

Stability of Embankments

An important consideration in the design or construction of any pier or embankment resting on a thick layer of soft clay is the possibility of shear failure occurring in the clay resulting in a slide of the embankment toward the toe of the slope. A number of such slides in piers, cuts and embankments have been reported in engineering literature. Perhaps the most famous of these in the slide which occurred during the construction of a quay-well at Goteborg, Sweden in 1916 and from an examination of which our present method of investigating the stability of slopes was developed.

Examination of a large number of slides has shown that failure of an embankment constructed in a clay soil or underlain by soft clay occurs by the sliding of a mass of earth along a surface which, in section, corresponds approximately to the arc of a circle, as shown in Fig. 6. Thus the entire sliding surface corresponds roughly to a section of the surface of a cylinder. This cylindrical sliding surface may pass above, through or below the toe of the slope depending on the extent and character of the clay.

Theoretically, whether or not such a slide will occur along any particular sliding surface in a relatively homogeneous clay may readily be determined. The weight of the soil mass located above the potential sliding surface tends to cause the mass to rotate about the axis of the cylindrical surface. This rotation is resisted by the shear resistance of the soil along the potential sliding surface. If the overturning moment due to the weight of the mass exceeds the maximum resisting

Fig. 1. Boston Harbor.

Fig. 2. New Mystic Pier No. 1.

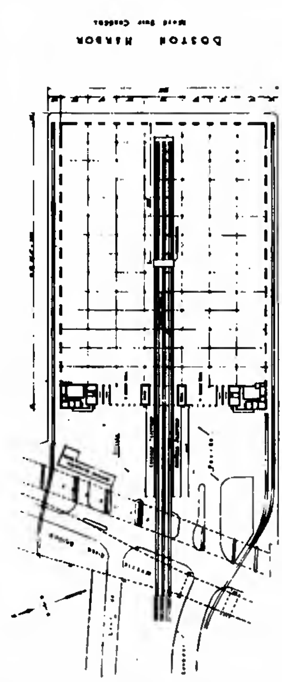

Fig. 3. Site plan of new Mystic Pier No. 1.

Fig. 4. Site of Mystic Pier No. 1.

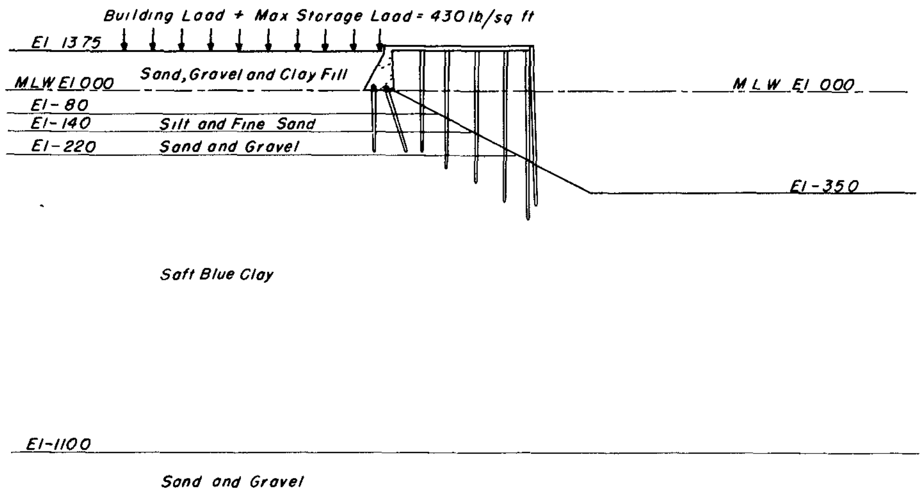

Fig. 5. Section through East end of old Mystic Pier.

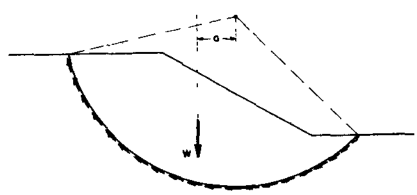

Fig. 6. Cylindrical sliding surface in clay soil.

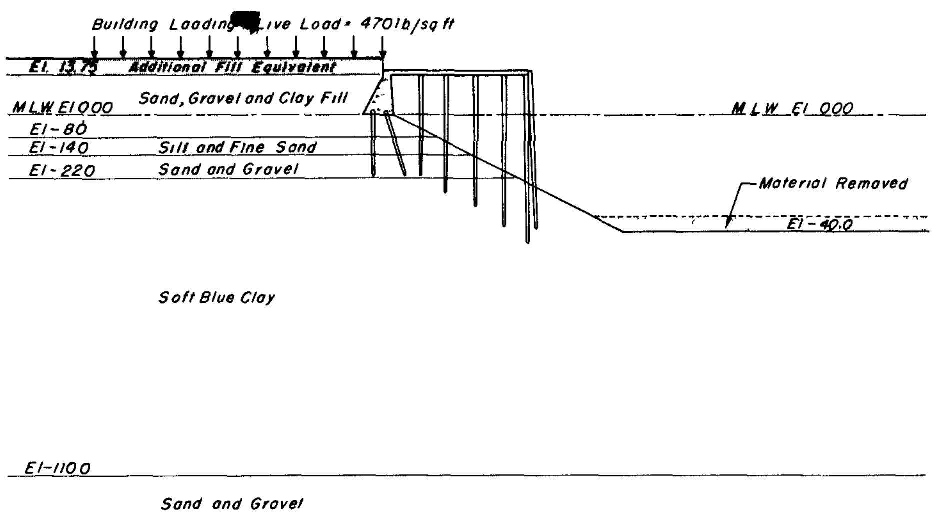

Fig. 7. Section through East end of pier showing effect of proposed changes on load distribution.

moment when the full strength of the soil is mobilized at all sections of the potential sliding surface, then failure will occur. The factor of safety against sliding along any particular surface is determined by the ratio of the maximum resisting moment to the overturning moment.

The main problem in analyzing the stability of a slope is that of determining the shear strength of the soil. A variety of test methods are available for this purpose, such as direct shear tests, unconfined compression tests, triaxial compression tests and field tests, and a variety of test procedures may be used. Unfortunately, these different tests methods and procedures give different results for the strength of a clay. However, experience obtained by the analysis of actual slides has shown that for slides in fairly homogeneous clay, the average shearing resistance along the sliding surface is roughly equal to one half of the unconfined compressive strength of the clay. Shear strength values determined in this way can therefore be used with some confidence for estimating the danger of a slide occurring in a fairly homogeneous clay.

Design Standards for New Pier

The design standards which were established by the Port of Boston Authority and which affected the selection of a suitable substructure for the new pier were as follows:

1. The ground elevation for the new pier should be raised to El. 17.0.
2. The deck of the transit shed should be designed to support a floor load of 600 lb/sq ft.
3. The channel around the pier should be dredged to El. -40.0 at the east end of the pier and to El. -35.0 along the north and south sides.

In itself, each of these requirements involves a relatively small change from the conditions existing at the old Mystic Pier. However, each of these changes tends to increase the possibility of a shear slide in the blue clay underlying the pier. The net effect of the changes corresponds to loading the entire area of the old pier with about 6 ft of compacted fill in addition to the freight houses and their storage loads and simultaneously increasing the channel depth by 5 ft, as shown on Fig. 7. The effect of such a change in load distribution on the stability of the pier is approximately equal to that produced by increasing the channel depth of the old pier by about 20 ft, as shown in Fig. 8. Such an amount of dredging has been known, in past experience, to cause failure of a previously stable embankment and thus considerable importance was attached to the problem of stability in the design of the new pier.

Stability Analysis of Old Mystic Pier

In order to ascertain the severity of the effects of the proposed changes, it was first necessary to make a stability analysis of the old Mystic Pier.

To determine the strength characteristics of the soils, two undisturbed sample borings were made close to the east end of the pier where it was thought the stability conditions would be most critical. A total of 20 samples of the blue clay, each about 2 feet in length, were taken in thin-wall seamless steel tubes using a piston type sampler. The samples were cut into sections which were tested to determine their natural water contents, liquid and plastic limits and unconfined compression strengths.

In general the samples were found to consist of grey-green slightly silty clay with some partings of silt and fine sand. The clay had a sensitive structure, characteristic of much of the clay in the Boston area; in the undisturbed state it was medium stiff and brittle but it became soft and sticky when remoulded at the natural water content.

A total of 49 unconfined compression tests were made to determine the variation of the shear strength of the clay with depth. Although there was considerable scatter of the results, the over-all average of the results for both borings showed a general increase in strength from about 0.6 kg/sq cm at El. -25 to about 1.2 kg/sq cm at El. -110.

On the basis of these results, a large number of trials were made to determine the surface along which the factor of safety against sliding was a minimum. This analysis, for the east end of the pier, showed the most critical surface to be located as shown in Fig. 9 with a computed factor of safety against sliding of about 1.15. For the north and south sides, where the channel depth was not so great, the computed factor of safety was slightly higher.

Such values for the factor of safety against sliding are somewhat lower than is generally considered desirable for earth banks of this type. However, in any such analysis a number of assumptions must necessarily be made and the computed result, while indicating the probable order of magnitude of the factor of safety, cannot be considered a precise determination. Furthermore, the old pier had been standing for many years and although its condition left much to be desired, it showed no signs of distress due to shear failure in the clay. Thus in spite of its low value, the computed factor of safety was evidently perfectly adequate. On the other hand, since the analysis was carried out using a method proved by experience to be relatively reliable, the computed result could not be considered to be too far removed from the actual conditions. A review of the assumptions made in the analysis showed that the computed factor of safety was unlikely to be too low by more than about 10% due to the use of assumed values; and even if the factor were 1.25 rather than 1.15, it was still as low as would normally be considered desirable for the design of a stable slope. Consequently, the conclusion drawn from this analysis was that although a factor of safety of 1.15, computed on the basis of the selected assumptions, was perfectly adequate, this value was also the lowest which could reasonably be adopted as a basis for the design of the new pier.

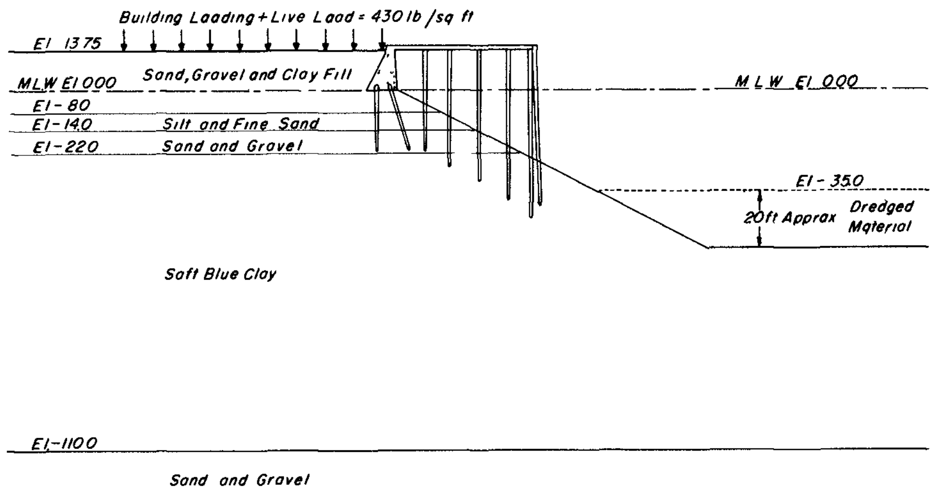

Fig. 8. Section through East end of old pier showing depth of dredging producing same change in stability as that caused by required additional load.

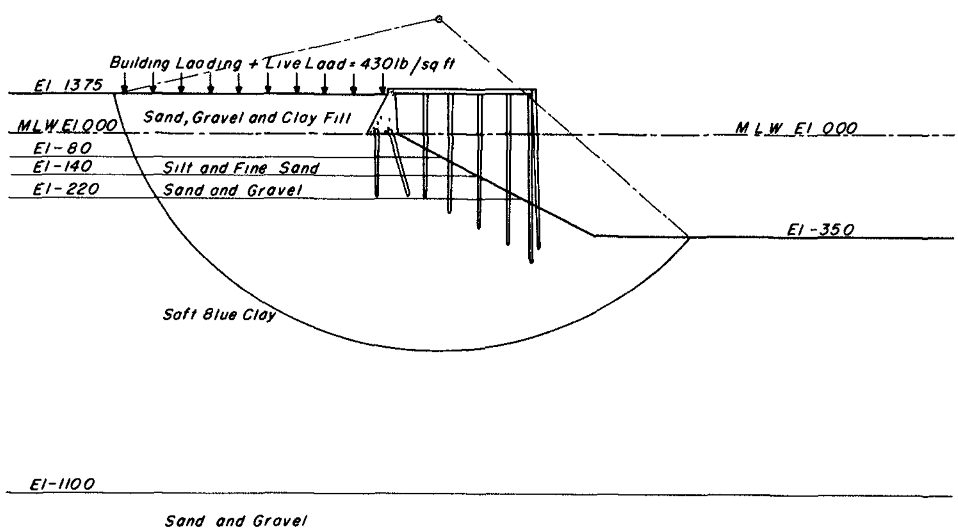

Fig. 9. Section through East end of old Mystic Pier showing most critical sliding surface.

Many factors were taken into account before this conclusion was finally reached. For example, it was known that in Europe the shear strengths of clays, in situ, had been measured by a vane shear apparatus and found, in the case of sensitive clays, to be considerably higher than the values determined by one half of their unconfined compression strengths. Therefore, it might be argued, the actual resisting moment was likely to be considerably greater than that used in the analysis and the actual factor of safety would thereby be considerably greater than the computed value.

To offset this were the results of recent research at Harvard University, where it had been found that under a sustained load, a sample of a sensitive clay would fail under a stress considerably lower than the normal compressive strength. Again, the development of progressive failure in a clay will cause the average shear resistance at failure to be somewhat less than that which would be produced if the full strength were mobilized simultaneously at all points in the mass. Thus even if the in situ strength of a clay were greater than the unconfined compressive strength, the effects of progressive failure and the application of sustained stress, as would occur in an embankment, would cause failure to take place at a stress lower than the in situ strength and therefore, at an average stress which might (1) be slightly greater than the unconfined compressive strength, (2) correspond closely to the unconfined compressive strength, or (3) even be slightly less than the unconfined compression strength. In the last event, the analysis would give too high a factor of safety rather than the conservative value indicated by consideration of vane shear effects alone.

However, the fact that in so many cases the average shear strength of the clay when a slide has occurred has been closely equal to one half the unconfined compression strength would seem to indicate that in general the combination of progressive failure and sustained load, and perhaps some other effect yet unknown, is just sufficient to offset the difference in strength between that of the clay in situ and that of a sample in the laboratory. It is because of this experience with actual slides that the method of determining slope stability using an average shear strength in clay equal to one half the unconfined compression strength can be used with a reasonable degree of confidence.

Effect of Proposed Changes on Stability of Pier

It has already been explained that the reconstruction of the new pier along the lines of the old one, and also in conformity with the design standards, would correspond to increasing the storage load of the old freight houses by about 650 lb/sq ft and increasing the depth of the channel by 5 ft. It was found that these changes would reduce the computed factor of safety against shear failure in the clay to about 1.03 at the east end of the pier and about 1.08 along the north and south sides. Since those values were below the minimum of 1.15 established on the basis of the stability analysis of the old pier, it became necessary to devise some alternative method of reconstruction which would not cause any reduction in the stability of the pier.

Selection of Substructure for the New Mystic Pier

The simplest method of relieving the load on the clay and yet meeting the design conditions was evidently to support the entire floor of the transit shed on a pile foundation. By using long piles driven through the clay, the weight of the structure and its storage load could be transmitted to the underlying layer of compact sand and gravel and the shear stresses in the clay would not exceed their original values. However, this method of construction would have required the use of about 1000 piles, each about 130 ft long, in addition to those used to support the deck of the pier,and it was quickly realized that the cost of such a substructure would be prohibitive.

Another method of relieving the load on the clay was to dispense with the fill required to bring the floor grade to El. 17.0 and instead, construct a concrete slab floor at this elevation, supported by beams and spread footings resting on the fill. However, the cost of such a floor slab to support a live load of 600 lb/sq ft was quite high and furthermore, there was considerable objection to leaving a space between the floor slab and the top of the fill on the grounds that it would provide a breeding place for vermin and therefore be unhygienic.

The design finally selected made use of a combination of methods for preventing any increase in shear stresses in the blue clay. First, the structure of the transit shed, but not the floor, was supported by long piles driven through the blue clay. This was actually a direct consequence of the pile support required for the concrete apron surrounding the transit shed. The piles used to provide this support were required to act, above the bottom of the channel, as columns and in some cases had an unsupported length of about 50 ft. Consequently, it was necessary to select a type of pile which would act also as a column and this requirement narrowed the choice to either steel H-piles or concrete-filled steel cylinders. Of these types of pile, steel H-piles were found to be best suited to the design conditions and also in this case, the more economical.

Another feature of the piles supporting the concrete apron was the high loads which they were required to carry. This resulted from the fact that their positions were determined to some extent by the locations of the numerous timber piles of the old pier. Some idea of the large number of timber piles, all of which were subsequently cut off below the water level, may be obtained from Fig. 10. In the new design, a careful survey was made of the locations of all these timber piles and, in order to facilitate construction, the new piles were located so as to minimize the danger of obstruction by the old ones during driving. It was felt that this careful procedure was rewarded during construction but it complicated the design by necessitating the placing of piles at different positions along the various bents so that a single standard design could not be adopted and by resulting in unusually high pile loads

Fig. 10. Timber piles in old pior.

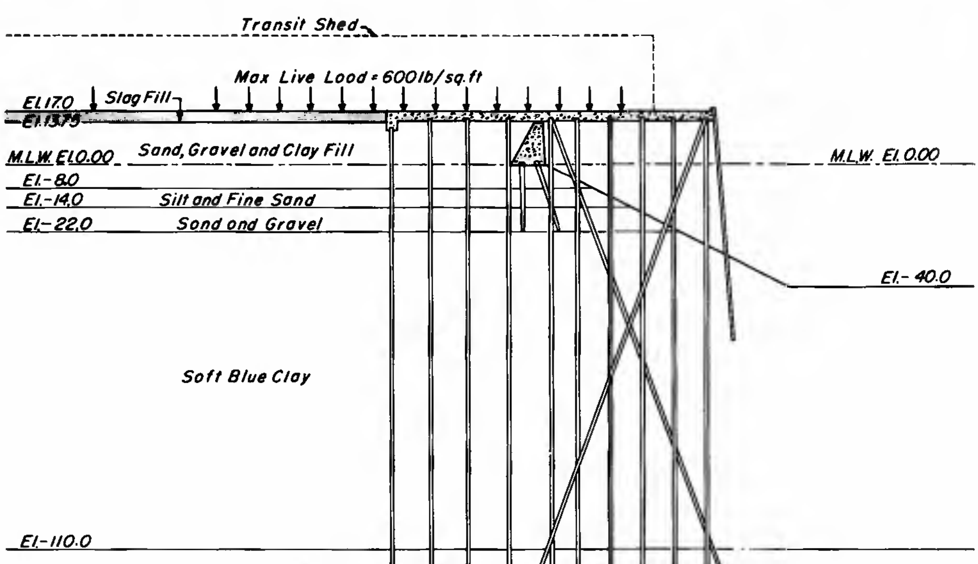

Fig. 11. Section through East end of new Mystic Pier No. 1.

where the positions of the old piles caused the new ones to be placed relatively far apart.

Because of their high loads, the piles supporting the deck had to be driven to a hard statum below the blue clay and since these piles supported also the edges of the transit shed structure, it was necessary to provide an equally firm foundation for the remainder of the structure. These long piles, then, served to prevent the weight of the structure itself from causing any increase in shear stress in the underlying clay.

The second method adopted to minimize the stress increase in the blue clay was that of using a light-weight slag fill to raise the ground level of the existing pier to the required elevation. By this means the weight of fill required was reduced by about 35 percent.

Since the two procedures described above were not sufficient in themselves to prevent a reduction in the factor of safety against shear failure in the clay under the new loading conditions, a further relief of stress in the clay was provided by extending the pile-supported concrete deck for a suitable distance behind the sea-wall. By this means the live-load at the edges of the pier was transferred to a bearing stratum below the clay. In addition, it was considered that the piles themselves would provide restraint against shear failure or plastic deformation of the clay. Thus the final design was similar to that shown in Fig. 11.

Analyses were made to determine what additional width of concrete deck should be supported on a pile foundation in order to maintain the computed factor of safety at its lowest permissible value of 1.15. In these analyses the chief problem was to evaluate the restraining effect of the piles. Since the piles are effectively restrained at their upper and lower ends, any tendency of the clay to deform will develop a bending action in the piles and, provided the clay can arch between adjacent piles without developing pressures exceeding the bearing capacity of the clay, the magnitude of the resistance provided will depend on the bending strength of the piles. Thus in order to determine this resistance it is necessary to make decisions on such problems as the lateral pressure distribution against the piles, the degree to which arching can develop in a clay soil, and the maximum permissible stress in the piles. Many hours of thought were given to these problems and the opinions of a number of leading engineers were carefully considered before the final decisions and analyses were made.

These analyses showed that it would be necessary to provide a pile-supported floor at the east end of the pier for a distance of about 50 ft behind the sea wall and along the sides, for a distance of about 20 ft behind the sea wall in order that the stability of the new pier should be essentially the same as that of the old one. The new pier was designed accordingly. It is believed that **for the required conditions, the design** provides adequate safety consistent with reasonable economy.

Perhaps the most interesting feature of this design is the fact that the major problem was not one of a really technical nature. When an engineer uses an accepted method of analysis to determine the stability of an earth bank which has been standing for fifty years or more, and he finds the factor of safety to be lower than or as low as is normally considered desirable, what should be his attitude towards making changes which will affect this factor of safety? It would probably be generally agreed that a factor of safety which has been adequate for fifty years will continue to be adequate for the next fifty years. But differences of opinion would exist as to whether any reduction in the factor of safety might be permissible. Some engineers would appear to believe that a long period of stability is indicative of a high degree of stability and that the height of a long-standing earth bank can be increased by, say, 20 percent without any undue risk of failure. On the other hand, there is the point of view that an earth bank which has existed for many years may be only just stable and may fail if the loading conditions are only slightly increased; experience shows this to have happened in many cases. Between these two extremes there is a wide variety of schools of thought.

Another important factor to be considered in such a problem is the cost of the measures required to prevent any reduction in the factor of safety. If the cost of the measures is excessive, it may be decided to take a calculated risk. Yet, even in this case, there must be a minimum permissible value for the factor of safety, and when this value is reached, the question of cost will become secondary since adequate safety must always be the primary consideration.

In the design of the new Mystic Pier No. 1 the designers were confronted with the dilemma of whether to place more faith in the intuitive opinions of reputable engineers or in the results obtained by a method of analysis which is necessarily subject to considerable limitations but which is based on previous experience of failures. It was finally decided that although the factor of safety was entirely adequate it was yet sufficiently low that no reduction should be allowed in designing the new pier. Fortunately, by a combination of measures, this was achieved at a reasonably small additional cost. It is believed that this approach, based on the technical advances in the field of soil mechanics in the past twenty-five years, was in good accord with the principles of sound engineering practice.

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The new Mystic Pier No. 1 was designed by Thomas Worcester Inc., Consulting Engineers, Boston for whom Mr. F. K. Perkins was chief engineer and Mr. M. Ja was project manager. Mr. George Wey, Chief engineer, represented the Port of Boston Authority on Technical matters pertaining to the design of the pier. The cooperation of these gentlemen in assembling the material presented in this paper is gratefully acknowledged. Thanks are also due to Mr. C. L. Monismith and Mr. L. C. Reese who assisted in making the stability computations and to Mr. G. Dierking who prepared the figures.