No. N-10

THE FOUNDATIONS OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE

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Probably no other modern bridge has presented the unprecedented and fascinating problems in foundation engineering encountered in the design for the San Francisco-Oakland Bay Bridge. The very earliest foundation studies indicated that conditions here would require pioneer design. (See Fig. 1) From an economic standpoint this bridge was required for the continued progress of the City of San Francisco. Situated at the northern end of a long peninsula lying between the Pacific Ocean and San Francisco Bay, its unusual advantages as a world port were each year more seriously handicapped by the requirement that shipments into and out of San Francisco must cross San Francisco Bay, over four miles of open water subject to seasonal fogs and other shipping hazards. Furthermore, a large proportion of the residential area for those employed in San Francisco lays in the East Bay Cities and Towns. The amount of traffic across San Francisco Bay may be gauged from the fact that the present ferry traffic between the thickly populated East Bay area and San Francisco's business section exceeds 500 ferry boats per day.

For these reasons there had been many studies to determine the possibility and feasibility of constructing a bridge from San Francisco to the East Bay area, but for many years the obstacles to such an undertaking seemed insurmountable, as for example--the great distance of open water from San Francisco to the nearest point on the East Bay shore; the extreme depth of water and the great depth to bed-rock in the West Bay section which seemed to preclude the assurance of an adequate foundation installation, and opposition of the War Department based on fear that a severe earthquake or enemy bombing of the bridge during warfare might cause the collapse of the bridge with a resulting closure of the harbor.

The Hoover-Young Commission was appointed in 1929, headed by President Hoover and the then Governor of California, C. C. Young, and including Rear Admirals Gregory and Stanley of the United States Navy, General Pillsbury and Lt. Col. Daley of the United States Army. C. H. Purcell, State Highway Engineer, was the secretary and Mr. C. E. Andrew, State Bridge Engineer, was associated. This Commission was the first body to overcome the objections of the Army and the Navy, clearing the way for a bridge project if found to be feasible of construction. After exploring several sites, the Commission selected a location extending from Rincon Hill in San Francisco, across Yerba Buena Island and then paralleling the Key Route Mole to Oakland. (Fig. 2) This site was carefully explored to determine the depth to rock in the West Bay Crossing, and the depth to suitable subsoil materials in the East Bay section.

The Hoover-Young Commission recommended, and the California Legislature authorized, the delegation of power to construct and operate the San Francisco Oakland Bridge, in the California Toll Bridge Authority. A Board of Engineers was appointed to determine the bridge design, and as a first step a comprehensive program of subsoil borings and soil tests was ordered. This program contemplated the procurement of sufficient information as to the character of the materials underlying San Francisco Harbor to permit

(1) A decision as to the best location for the bridge
(2) The determination of the best locations for the individual piers, and
(3) The determination of soil bearing values and other soil factors affecting pier designs.

The Board had access to the jet borings made by the Hoover-Young Commission covering a wide area of the Bay. These borings, while made primarily to determine the depths to bedrock, included frequent samples of the overlying soil, and a contour map made from the boring records clearly indicated the selected route as approximately the most advantageous. In order to locate accurately the crest line of the rock ridge, a considerable number of wash borings were sunk to rock within an area extending north and south from the tentative location recommended in the Hoover-Young report. These wash borings confirmed in general the findings of the Hoover-Young Commission and resulted in the final selection of a center line location of the West Bay crossing slightly to the south of that originally proposed in the Hoover-Young report.

After a thorough study of various types of bridge structures, the Board agreed upon a design for the West Bay crossing comprising a double suspension structure with a central anchorage. (Fig. 3) Additional dry sample borings were then sunk at each approximate pier location to facilitate exact pier location, and to determine the depth to and character of rock at each pier and the conditions to be encountered in sinking the deep caissons to rock. Extensive rock cores were obtained by Diamond Drills to accurately determine the character of the bedrock.

These borings developed the fact that the caisson piers in the West Bay crossing must be designed to float during sinking to a maximum depth of 120 feet, and must be sunk to bedrock a maximum depth of 240 feet. (Fig. 4) The Mississippi River Bridge at New Orleans was then in process of design and contemplated piers to the then unprecedented depth of 180 feet, with maximum depth of 75 feet of water. The deepest piers ever installed up to this date were those for the Hawksbury Bridge at New South Wales, Australia, where a maximum depth of 162 feet had been reached, with a depth of 70 feet of open water. The San Francisco Bay Pier installation, therefore, contemplated sinking piers to a depth 50% in excess of that previously accomplished, with a flotation depth 70% in excess of the Hawksbury installation.

The foundation problem for the East Bay crossing, from Yerba Buena Island to Oakland, differed radically from that of the West Bay crossing. Borings showed that rock drops off sharply immediately

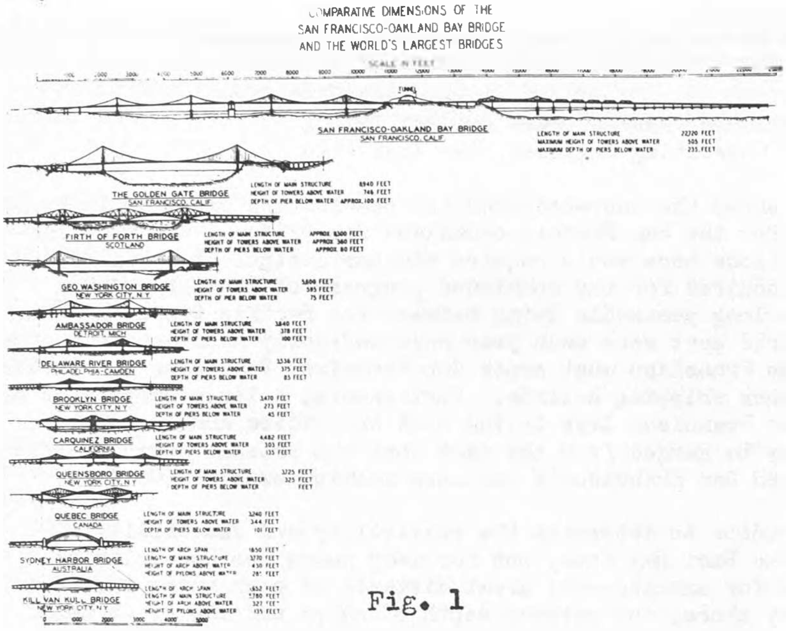


Fig. 1

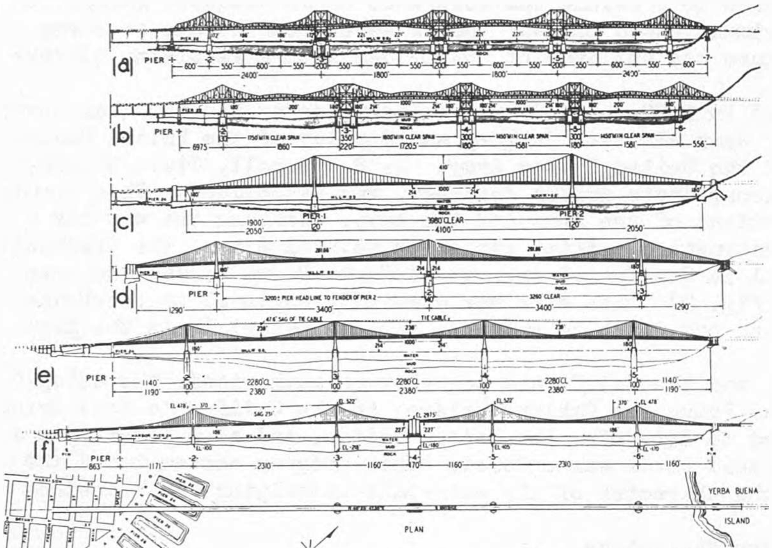


Fig. 3

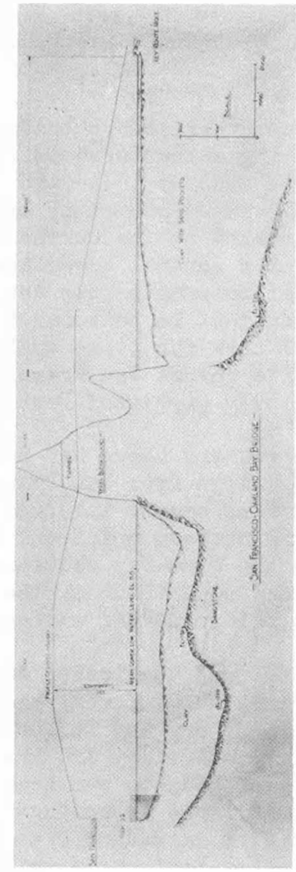


Fig. 4

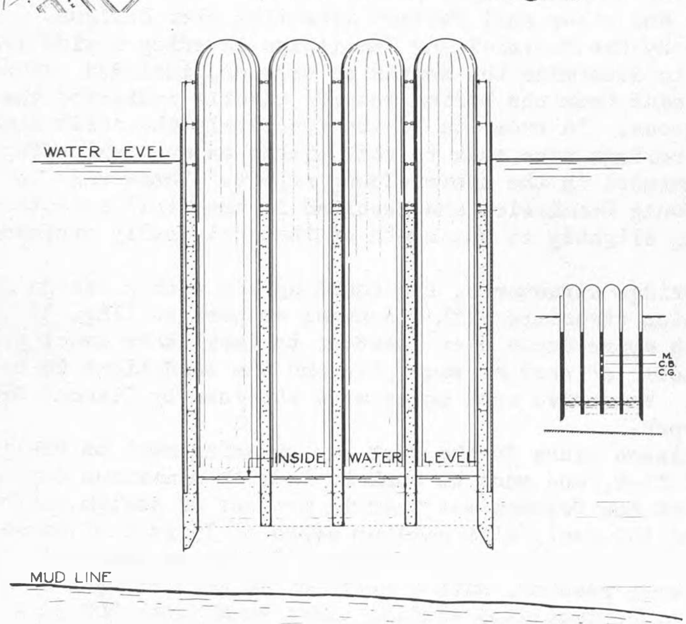


Fig. 8 MORAN CAISSON-FLOATING

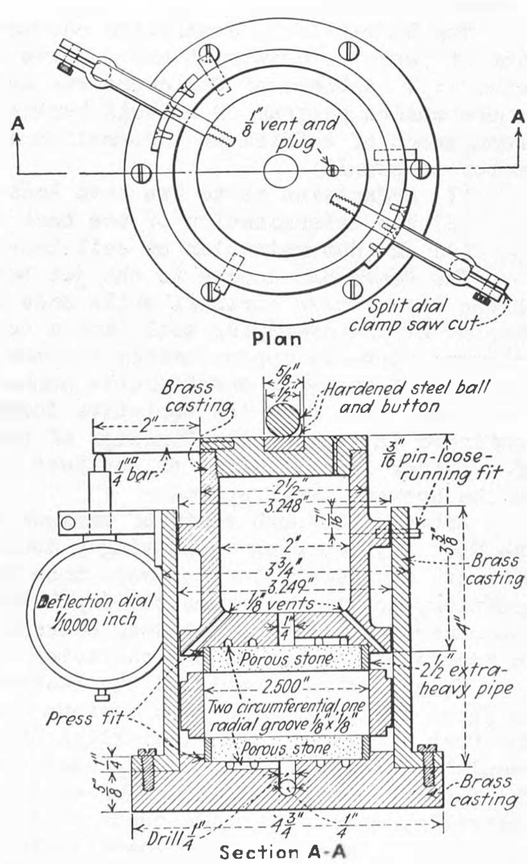


Fig. 5

east of the Island, to a depth which is unreachable by any practicable method of construction. For these foundations, therefore, it was necessary to determine accurately the character of the soil so that the piers might be designed to bear on an adequate subsoil stratum at safe intensities of load. In order to obtain the information required for a complete analysis of the soil, a sampling device was developed for this project, as described in the Engineering News-Record of June 23, 1932.

Soil Tests. Undisturbed samples were hermetically sealed on the drill barge, immediately after their procurement, and shipped to the laboratory of the University of California for testing. Duplicate samples were sent to the Moran and Proctor Laboratory in New York for comparison and testing. In the laboratories, determinations were made of the structural characteristics and behavior under load of the soils to be loaded, and the character of the strata above the foundation levels which might affect problems of caisson sinking, excavation and construction. A complete series of routine tests were made including:

(1) The specific weight in the undisturbed condition
(2) Moisture content
(3) The specific gravity of the constituent grains
(4) Sizes of the constituent grains by the use of the Bouyoucos hydrometer, separating grains down to 0.001 mm.
(5) Atterberg's shrinkage, plastic and liquid limits and undisturbed shrinkage.
(6) Compression tests to failure on 2" x 2" cubes and on cylinders of undisturbed soils without lateral restraint.
(7) Compression tests on 4.87" x 9" cylinders of undisturbed materials under lateral restraint, in order to evaluate the effect of lateral restraint on the compressive strength and deformation of the clays.

In addition to these routine tests, consolidation tests on both undisturbed and remoulded samples were made to determine the design unit intensities and the probable settlements. The apparatus used in making the consolidation tests was especially designed for this work and was believed to constitute an improvement over devices previously used. (Fig. 5)

For the design of pile foundations in the East Bay crossing, the information obtained from the consolidation tests on remoulded samples, together with that obtained from a loading test on a group of nine piles provided valuable data on the ultimate bearing capacity of the piles, and permitted the engineers to predict within reasonable limits the probable settlements and rates of settlements of the pile foundations.

In the case of Bent No. 37 alone, the settlements have been greater than expected. At this location, the generally uniform stratification of materials seems to have been cut by a deep channel with a soft refill. This condition would have been disclosed by a boring at the exact site of this pier, but at the time the borings were being made financial limitations made it necessary to limit the number of borings and consequently this location was not included in the boring program.

Caisson Design. The principal construction problem lay in the design of the West Bay piers. As the depth of open water and the depth of required sinking to rock were far beyond the limit of pneumatic work, the only available method of installation was by open dredging caissons. Also, the piers must have large horizontal dimensions because of the bridge loads and the very heavy lateral forces caused by winds, currents, earthquake impulses, etc. (Fig. 6)

The success of the design very evidently centered around the problem of controlled caisson flotation, as the caissons must be considered in full or partial flotation until their cutting edges are buried in subsoil materials sufficiently firm to support their dead weight. All previous open dredged caissons had employed either the braced cofferdam principle or the false bottom method. The false bottom method was quickly dropped from consideration because, to resist a 120 ft hydrostatic uplift, it must of necessity be extremely heavy and difficult of removal, and involve such hazards and risks of removal as to seriously jepardize the probability of safe installation. The inrush of subsoil materials to be anticipated with the removal of each false bottom at such depths as required here might very probably undermine the support of the caisson and cause a sudden tilting and submergence, and possibly the loss of the caisson.

The braced cofferdam principle of flotation requires an open cofferdam from 1/2 to 3/5 of the depth of flotation, and such a cofferdam, to resist the hydrostatic pressures at these depths, would be extremely heavy and expensive to build. It would be objectionable because of the weakening effect on the permanent pier caused by the reduction in pier area necessitated by the veritable forest of cofferdam bracing which would be required.

The more this problem was studied, the more evident it became that a new principle of foundation installation must be developed to meet the difficult conditions. After months of study of many different schemes, there was evolved the type of caisson which has since become well known as the Moran Caisson. (Fig. 7) Mr. Daniel E. Moran conceived and patented the idea of constructing a cellular caisson with pneumatic flotation, which would provide the equivalent of a false bottom under each dredging well, where such false bottoms could be moved at will upward or downward within the dredging wells and could be ultimately removed preparatory to the open dredging of the caisson. Also the Moran Caisson provided for the final decompression of the dredging wells under complete control and accompanied by the introduction of water to replace the pneumatic column, until adequate support for all cutting edges was provided by the underlying soils.

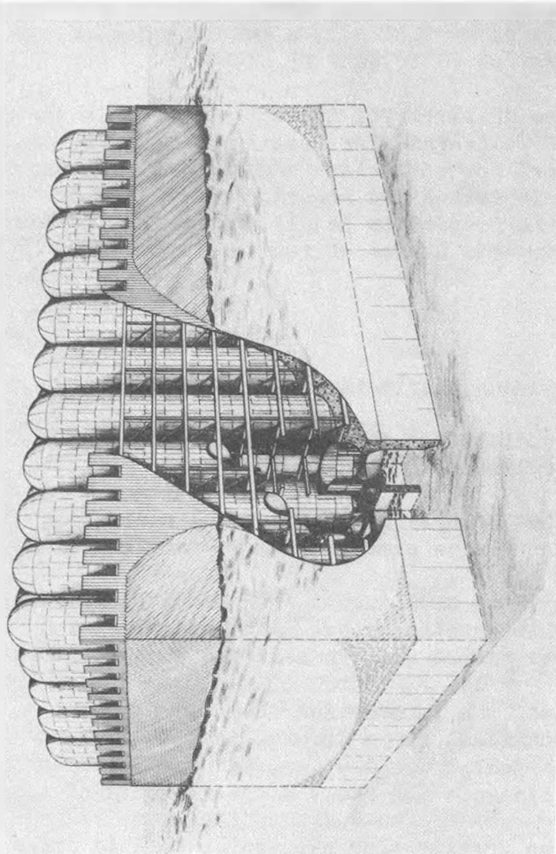


Fig. 7

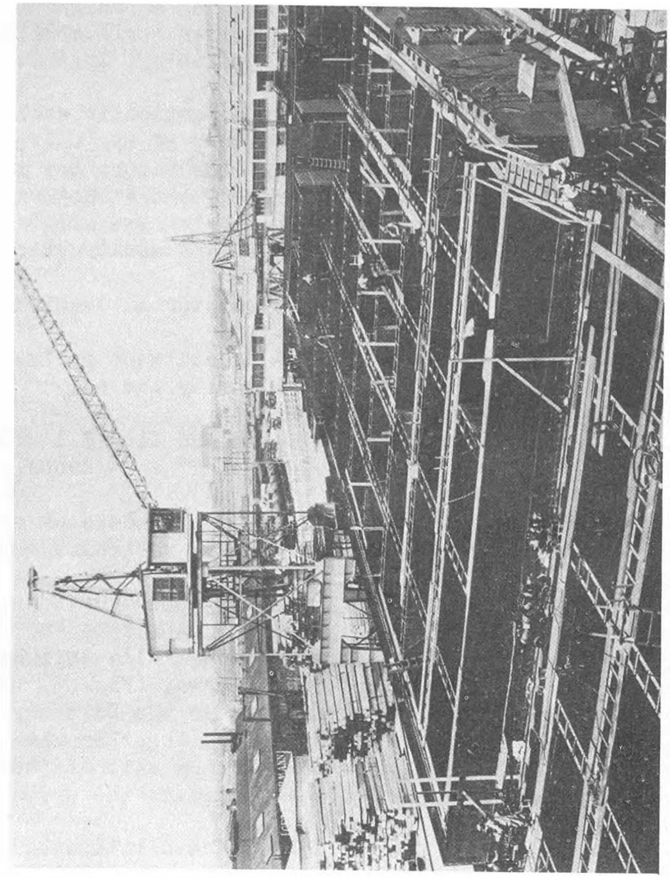


Fig. 11

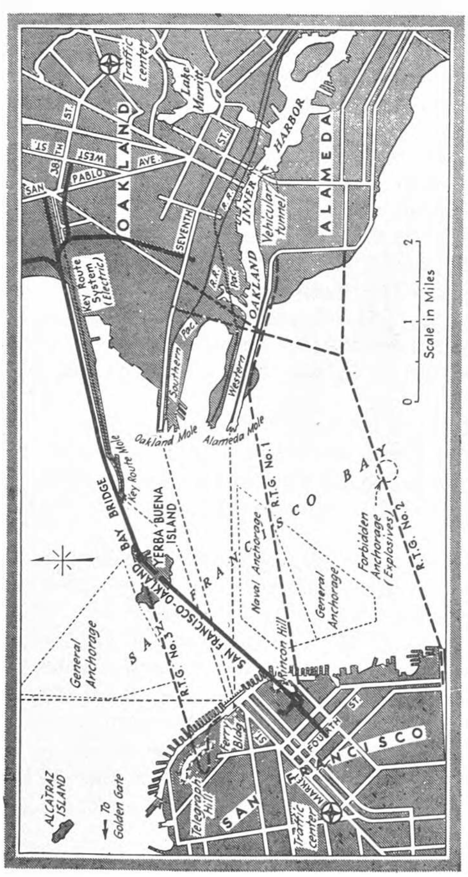


Fig. 2 ROUTE OF NEW BRIDGE over San Francisco Bay, showing also three alternate locations proposed in 1927 by the Ridgeway-Talbot-Galloway board of engineers.

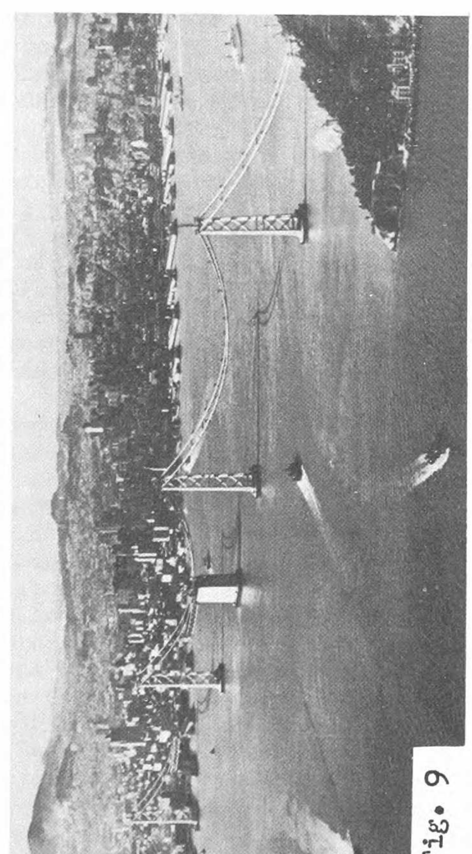


Fig. 9

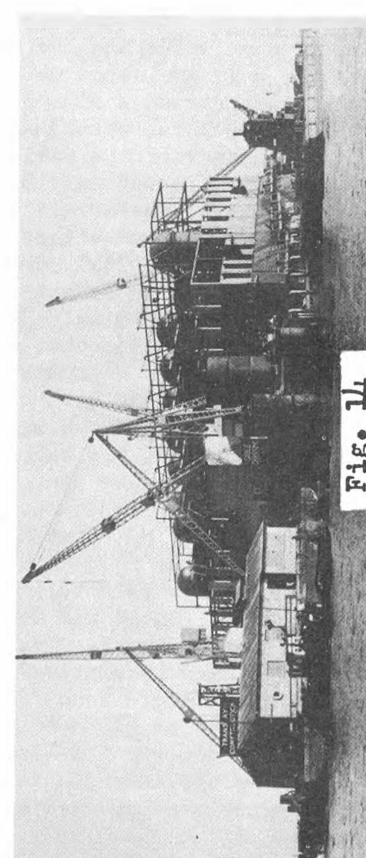


Fig. 14

This was accomplished by constructing the dredging wells as circular steel cylinders, which were extended well above the top of each caisson build-up and covered by steel domes fitted with valves and connections for the introduction of compressed air and for the later introduction of water as the compressed air was blown off. By properly proportioning the area of air-filled dredging wells to that of the caissons, required flotation is obtained; but in order that the center of gravity of the caisson should be always well below the metacenter, providing a positive righting moment and preventing the caisson from tipping, the caissons were provided with low head cofferdams in conjunction with the air-filled cylinders.

Not only did the development of the Moran Caisson assure the success of this project, but its influence on future bridge projects may be such as to make possible and economically feasible, bridge pier installations which might otherwise be economically or physically impossible. Assuming the depth to the river bottom increasing as the distance increases from either shore, previous methods have required, for economy and feasibility of pier installation, that the pier location be controlled by the maximum economical depth of caisson flotation rather than by the depth of caisson dredging. The unit cost increased and the economy decreased rapidly with increased depth of flotation. The exact reverse holds for the Moran Caisson, where the unit cost of installation decreases with increase in depth of flotation. Therefore, bridge piers in the future may be installed in greater depths of water than heretofore and bridges may be designed for maximum economical spans.

The Moran Caisson as designed provided 10% excess in flotation effect, so that 10% of the domes could be removed at one time to permit building up the cylinders while the caisson was being sunk to bottom. (Fig. 8) The caisson was designed to float with the water level within the wells not less than 10 feet above the cutting edge. For the San Francisco Bay caissons it was assumed that the cutting edges would penetrate into the semi-liquid mud bottom to a depth of 15 feet before the weights of the caissons were transferred to the soil.

With these basic assumptions established, comparative studies were made involving various cylinder sizes and wall thicknesses. Each study required a thorough investigation of flotation and stability, bracing and cylinder design, stress determination for the final pier and estimates of cost. From these studies was evolved the final design which proved economical, thoroughly practicable in each stage of installation and flotation, and the fulfillment of every hope in the reduction of usual hazards and contingencies.

In addition to its positive control of all stages of flotation, this caisson has the additional outstanding advantage that, as it approaches the bottom, it may be accurately located and quickly lowered to a firm embedment in the underlying soils. Ordinary methods of caisson installation require the gradual approach of the cutting edge to the mud bottom, and therefore in swift currents there may develop a scour under the cutting edge as its distance from the soft mud bottom is reduced. This method provides the further advantage that whenever the cutting edge is within an allowable pneumatic working depth, air locks may be installed on the domes, and workmen may enter the cylinders to remove obstructions at the cutting edge level. When the domes are in use they serve as cushions against the caisson listing, since the effective center of gravity is lowered, and the load on the cutting edge is reduced by the compressed air in the wells. When sealing the caisson, air pressure may be used to reduce the effective weight on the cutting edge.

Construction Started. The West Channel crossing, between San Francisco and Yerba Buena Island, includes two - 2210 feet suspension spans with 1160 feet side spans, supported by 29" diameter cables. Fig. 9. A common central anchorage, 97 feet x 192 feet in plan, and which extends from elevation minus 220 to plus 295, supports the unbalanced live loads on the two suspended spans.

The crossing through Yerba Buena Island is the largest bore rock tunnel ever attempted, the excavated inside width of the tunnel being 79 feet. (Fig. 10) The East Bay crossing includes a 1400 ft cantilever span immediately east of Yerba Buena Island, followed by five 509 ft spans and fourteen 291 ft truss spans.

For the West Bay caissons, the cutting edge sections were fabricated, ready for towing to the site, at the shipyards of the Moore Dry Dock Company in Oakland. (Fig. 11) The cutting edges consisted of hollow intersecting box girders made of steel plate, with joints welded to form watertight compartments. The cutting edge girders were arranged in 15 ft square grid patterns, each square centering under a steel plate dredging cylinder. The transition from the 15 ft square to the 15 ft diameter cylinders was accomplished through adaptor sections of welded steel plate. Steel wales and struts were carried through between the cylinders to support the exterior timber skin of the caissons.

Since each cutting edge section must be towed to its pier site before the installation of concrete in the walls or deck, each assembly was designed as a boat hull. (Fig. 12) While being towed to its site and during its entire period of flotation each caisson must:

(1) Support the excess weight of the exterior cutting edges and side walls and distribute these weights over the entire caisson area.

(2) Resist the action of heavy swell or waves.

(3) Support and distribute concentrated weight of green concrete during the placing of the concrete build-ups.

Placing the Caissons. Because of the swift flowing tide, the physical characteristics of the site, the tremendous amount of ferry traffic and the large number of ocean going ships transversing the line of the bridge, and the heavy fogs on the Bay, it was very important that the design of the foundations and

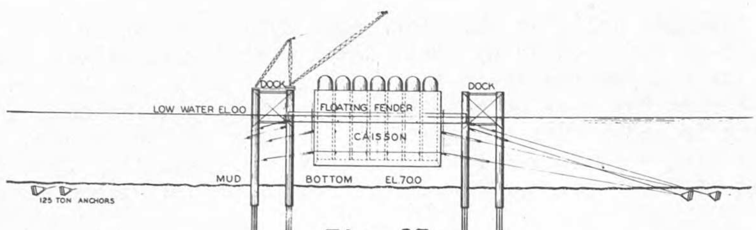


Fig. 13

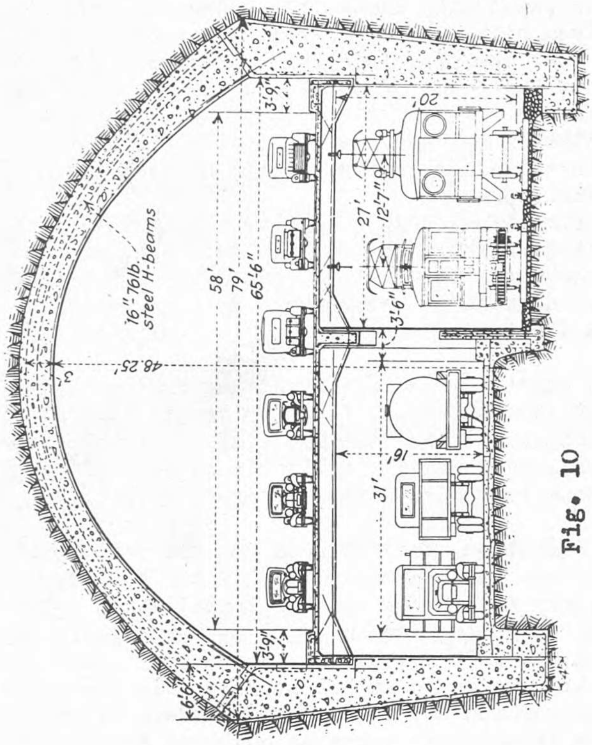


Fig. 10

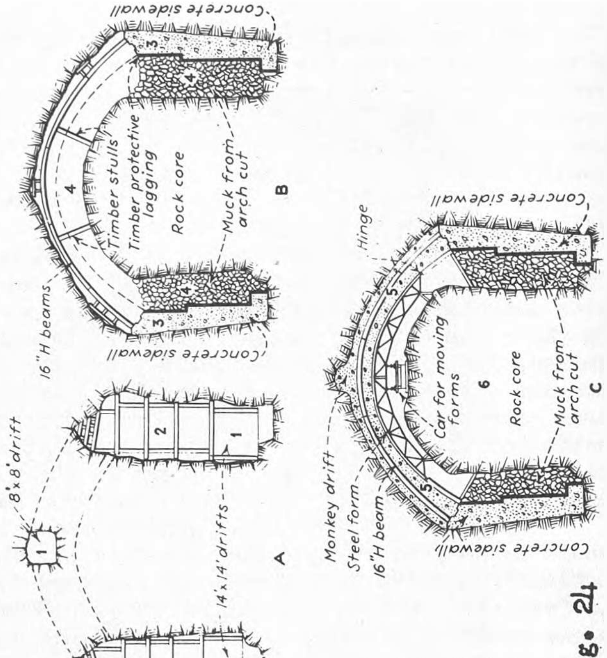
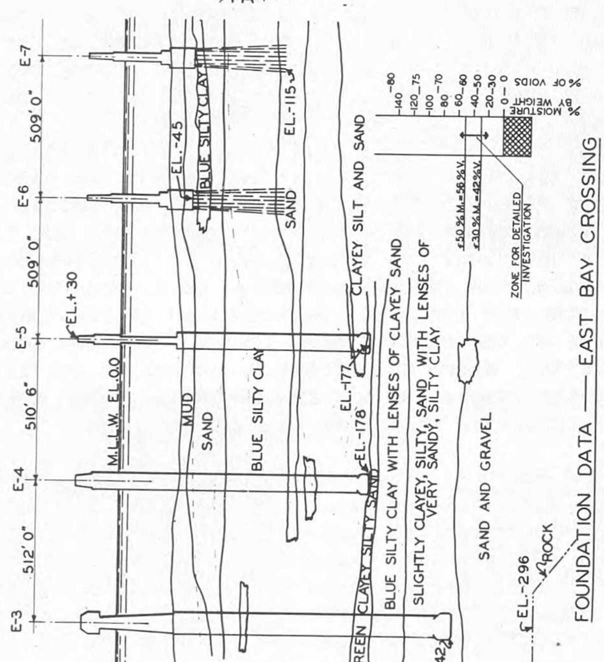


Fig. 24



FOUNDATION DATA — EAST BAY CROSSING

Fig. 21

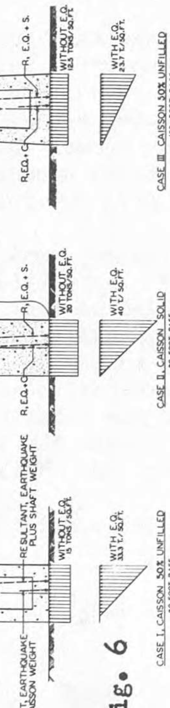


Fig. 6

RELATIVE EFFECT OF INCREASED WEIGHT AND INCREASED BASE WIDTH ON STABILITY OF BRIDGE PIER SUBJECT TO EARTHQUAKE FORCES
EARTHQUAKE FORCE ASSUMED = 5% OF GRAVITY

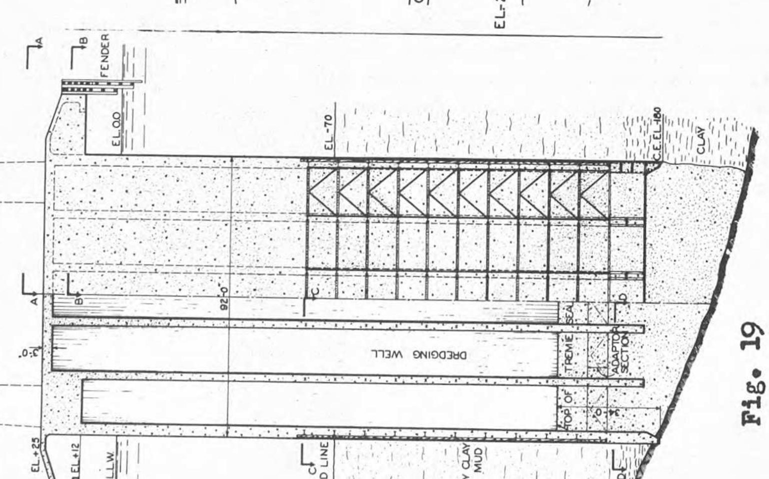


Fig. 19

the construction procedure be such as to minimize the hazards of installation. Providing adequate fender protection against collision, and adequate support for the working equipment was a major problem in this installation. (Fig. 13) Caissons were surrounded by massive floating timber fenders outside of which lay construction equipment. In addition to floating equipment, docks for the support of construction equipment were installed consisting of large diameter steel cylinders supported on timber piles. The cylinders were concreted and were interconnected by a steel bracing system. A system of 20 ton concrete mushroom anchors was provided to anchor the caisson and the floating fender.

When each caisson reached its site, its anchor lines were attached and adjusted and additional lengths added to all cylinders, after which the domes were welded to the tops of the cylinders. Air was then pumped into the cylinders so that the water was depressed to the desired depth above the cutting edge and the air pressure was maintained so as to hold the caisson at an elevation where the side wall cofferdams were not overstressed. Anchor lines were run from the 20 ton reinforced concrete anchors to connections at high and low levels on the caisson, such connections being provided at the ends of the trusses running through the cross walls between the cylinders. The anchor adjustments were made through load lines running up the sides of the caisson to handcrabs, where cable pulls up to 125 tons were developed. The lever arm ratio of the handcrab from the load line was about 100 to 1 and the load line led to a 5 sheave block providing a 10 part line to the anchor. Deflection readings taken 3 times a day determined the exact pull in each anchor cable.

Each caisson was then concreted and the side wall cofferdam continued upward until the caisson submergence required extensions of the steel cylinders. The cylinders were extended upward by burning off not over 10% of the domes at one time, welding on cylinder extensions, and replacing and welding on the domes. (Fig. 14) To facilitate these extensions, the domes were constructed with 24" long skirts, each burning operation cutting off about 2" of the skirt. When each caisson approached mud bottom, the top of the concrete wall was about 21 feet below water level. The actual grounding of the caisson required great care and delicacy of handling. With the amount and direction of tidal current acting on the caisson carefully measured and studied, the most favorable hour for minimum tidal action was selected and the anchor lines adjusted for sag and for caisson movement with tidal changes. At an exact predetermined time, the caisson was lowered into the mud, the position of the caisson being constantly observed and the anchor line adjustments progressively made. By the rapid release of air the caisson was quickly lowered 2 or 3 feet into the mud, and after a definite check of its location, it was allowed to sink into the mud a depth sufficient to fix its position definitely, usually 8 or 10 feet.

Excavation Begun. The lower anchor lines were then detached and some of the domes removed so that while dredging was being carried on through some wells, the remaining domes still contributed to the stability of the caisson. (Fig. 15) In some cases, where there was a sloping hard stratum below the top soft mud, the tendency of the caisson top to follow the inclindation of the hard stratum was counterbalanced by domes which were left in place until the cutting edge penetration provided equal support on all edges.

For controlled landing and initial sinkage into the mud, it was necessary to know the water level and the mud level within the cylinders before removal of the domes or full release of the air. By an ingenious method, the mud level and the water level within any cylinder was determined at any time. An electric cable attached to a weighted split contact point was placed in a gasketed pipe sleeve about 4 feet long. A tap was removed from the top of the dome and piping immediately installed, the gasket preventing appreciable loss of air. (Fig. 16) The cable was then fed into the pipe, the weight pulling it down until the split contact point reached the water surface when a lamp connected to the cable circuit lighted up to indicate the elevation of the water. The weight was then dropped further until the mud was encountered.

With the caisson firmly supported on the soil, all domes were removed and dredging carried on progressively through all wells. The side wall cofferdam and concrete were carried up as sinking continued. As the caisson approached rock, it was frequently necessary to jet under the cross walls, and thorough washing of the rock bottom was required, before final inspection of the rock floor and the placing of the concrete seal. A novel method for jetting under the cutting edges was developed. The arrangement consisted of a frame supporting a jet by a toggle joint at the center. (Fig. 17) The frame fitted closely into the cylinders and resisted the kick-back of the jet pipe while the toggle joint permitted easy movement and control of the jet.

Construction Difficulties. The contract for the West Bay foundations was awarded on April 9th, 1933 to the Transbay Construction Company, and the total cost amounted to 7-1/4 to 7-1/2 million dollars. The contract required the installation of five piers designated as W-2 to W-6 inclusive. W-2, which was the first pier off the San Francisco shore, was a steel sheet piled cofferdam type of installation, made possible at this location because of the relatively shallow depth to bedrock, but W-3 to W-6 inclusive were Moran type caissons. W-4 was the largest caisson in plan dimensions ever installed, being approximately 100 feet x 200 feet in size.

W-6 was the first caisson placed by the contractor. After the cutting edge had entered the mud about 10 feet, the contractor prematurely removed all domes and started dredging. With a weight of 55,000 tons the top of this caisson suddenly tipped 16 feet to the east within a time period of 10 seconds on January 16, 1934. Materials underlying the caisson cutting edges were firm enough to hold the caisson in position after its initial tilt, and after re-installation of the domes on the low side and dredging through wells on the high side, the caisson was righted.

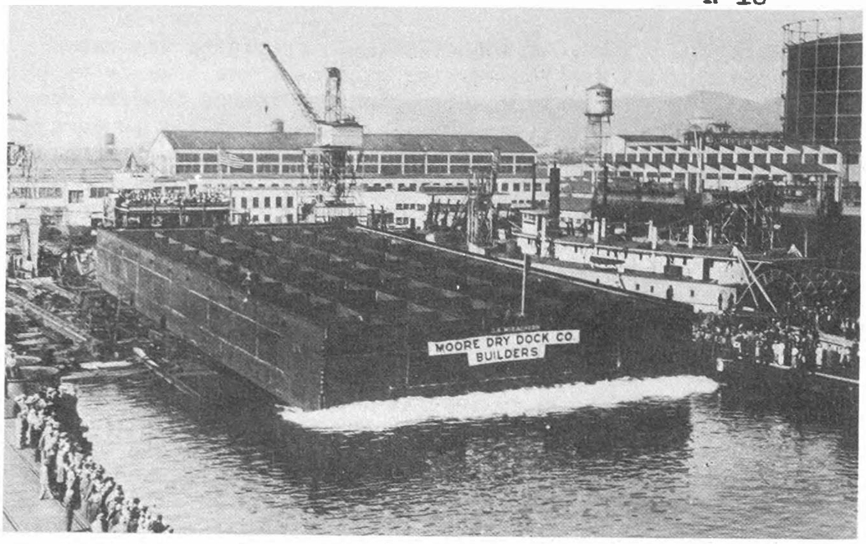


Fig. 12

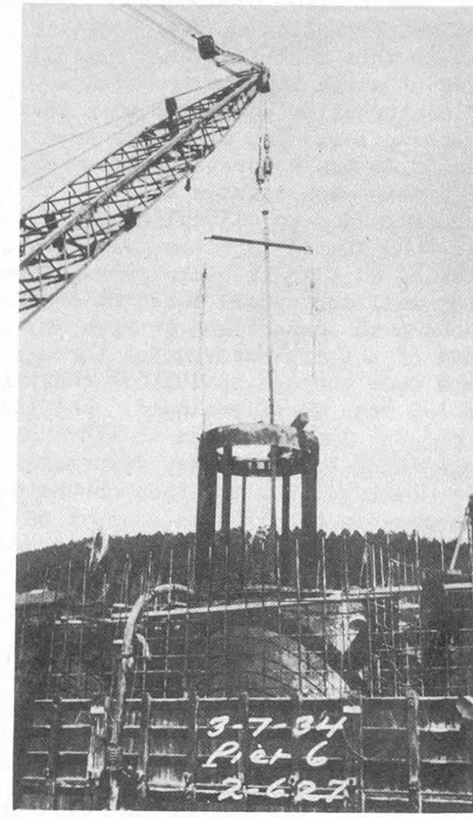


Fig. 17

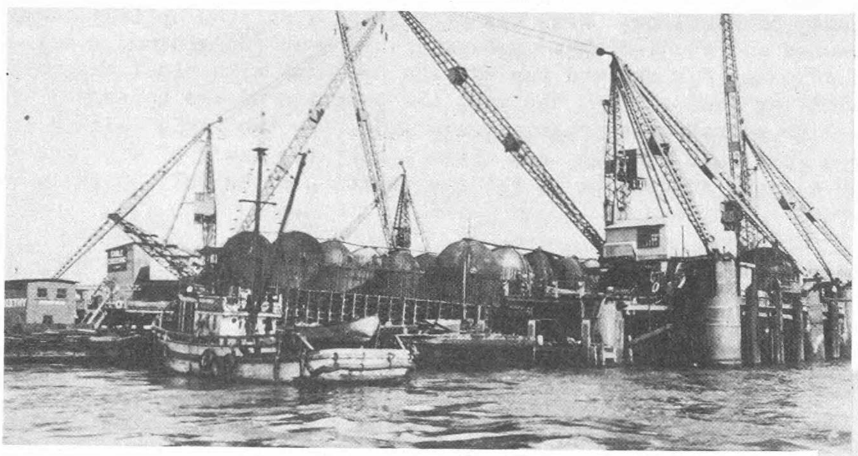


Fig. 18

Fig. 16

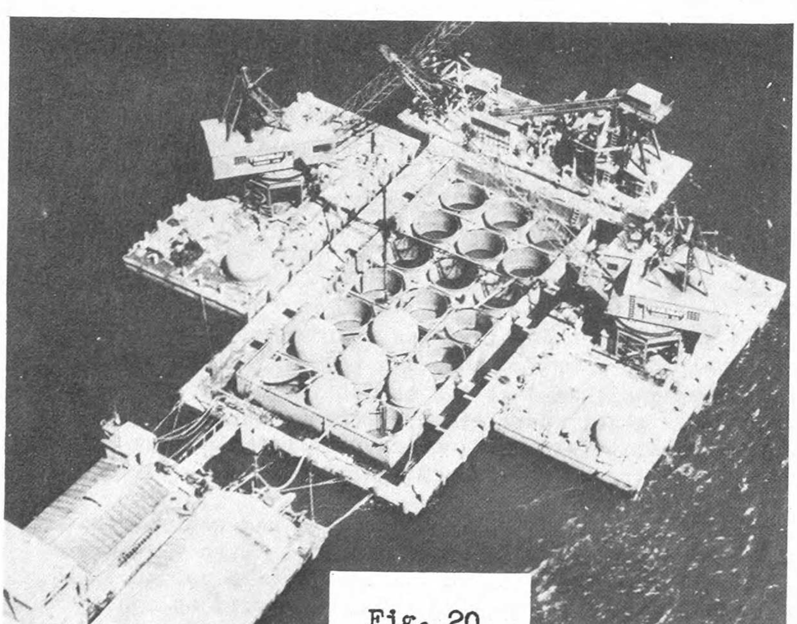


Fig. 20

When caisson W-4 was landed on a subsoil radically different and much softer than that under W-6, the contractor again removed all domes, when the entire caisson weight together with the weight of the water and soil contained in the wells was supported by the underlying materials. This caused a lateral movement of the underlying material, partially wrecking the pile supported working platform, and causing a mud wave and upward movement of soil material around the caisson. On January 31st this caisson, with a total weight of caisson alone of 75,000 tons, started to tilt to the north and continued to tilt at the rate of approximately 1 inch per hour until February 7th when a lateral movement of 16 feet at the top had occurred and when the north edge of the caisson cofferdam was submerged at high water. (Fig. 18) The methods which had proven successful in the righting of caisson W-6, when applied to this caisson, only tended to increase the tilt. The materials underlying this caisson were so soft that they forced themselves into the dredging wells to a maximum distance of 65 feet and were acting as plugs or false bottoms to the dredging wells. By removing these plugs of soil materials from the second row from the north or lower end of the caisson, a flow was induced under the plugs in the rows of cylinders to the south and on February 12th, the caisson began to move back towards its normal vertical position. However, the materials were so soft that when a position of verticality had been reached, it was impossible to check the rotating movement of the caisson, and the caisson continued to move in a southerly direction until the south end had dropped, and the caisson had tilted to the same extent which had previously taken place to the north, i.e. 16 feet. During this rotational movement to the south, there had been enough sinking of the caisson to permit the extension of the cofferdam and to make it more effective, which with the reinstalment of a portion of the domes, the removal of the water from the upper portion of the domed wells, and the careful dredging of the wells, brought the caisson back to level and under control and it was held in its normal position thereafter through the balance of its sinking operations.

The experience gained by the contractor on W-6 and W-4 resulted in his maintaining on subsequently sunk caissons adequate cofferdams and a sufficient number of domes until adequate penetration into the soil assured a fixity of position, and no difficulty in sinking or tilting was experienced in any of the other caissons. It may well be asked why the contractor was allowed to remove the domes prematurely. The contract placed the responsibility for the handling of the caissons on the contractor, and direct orders as to the method he should follow, would have at least partially relieved him of this responsibility. Only after learning through his own experience was he willing to follow the Engineer's advice that he maintain adequate cofferdam and domes in position in order to reduce the weight on the underlying material by the removal of the entrapped water.

Caisson W-4 landed on a rock bottom which sloped off to such an angle that the high point of rock under the cutting edges was approximately 30 feet above the low point. (Fig. 19) It became necessary to provide support for the portion of the caisson where the rock was deep, while the rock excavation was carried on until all cutting edges received their support in rock. This temporary support was provided through the reinstallation of domes on the cylinders over that portion where the rock was deep. The large amount of such underwater rock excavation involved a run-in of outside materials and a very thorough cement grouting operation to seal the ground and check the inward flow so that the bottom could be prepared and the caisson concreted to bedrock.

Caisson W-5 penetrated only a short depth of mud before landing on an inclined rock surface. Here it proved advantageous to carry air pressure on part of the cylinders during excavation and also during the concreting of the bottom. (Fig. 20) Under these circumstances it will be readily recognized that either a false bottom or a floating braced cofferdam caisson would not have been practicable or economical to meet the difficulties encountered in flotation, landing and sinking.

East Bay. For the East Bay crossing the foundation design differed radically. Here three types of foundations were required by the extreme variation in load and in soil conditions at different pier locations. The maximum loads in the East Bay crossing are under the two piers supporting the long cantilever span. Pier 2, immediately east of Yerba Buena Island was founded on rock easily reachable through a steel sheet piled cofferdam, but the rock pitches off very rapidly east of Yerba Buena. Under Pier E-3 rock is at -290 feet but an adequate foundation stratum of compacted sand and gravel was found at -240. The load intensities and the subsoil conditions under Piers E-4 and E-5 required them to be carried deep to firm supporting strata; the remaining Piers in the East Bay crossing are supported on long timber piles.

The geological cross-section under the principal East Bay Piers shows the subsoil materials classified on a basis of moisture content or per cent of voids. (Fig. 21) Many soils laboratory analyses and tests were required to determine accurately the depth, bearing area, allowable load intensity, etc., for these piers, and this work was simplified and expedited by concentrating detailed investigations on soils in the lower brackets of moisture content.

Piers E-3, E-4 and E-5 consisted of false bottom reinforced concrete dredging caissons. The contractors bidding on the work had the option of selecting the false bottom or the sand island method for building these foundations, either being suitable because of the shallow water at their sites. A fairly heavy caisson was needed to overcome the skin-frictional resistance, particularly of Pier E-3, which had to be sunk to a depth of over 210 feet below the Bay bottom. After the false bottoms are installed they are braced to the side walls and the caisson made ready for the side wall build-up prior to towing to its site. (Fig. 22) The reinforced concrete walls forming the dredging wells are then built up and the caisson sinks as weight is added until it lands on the mud bottom, when the false bottoms are removed. To reduce the pier weight, and thus the intensity of load on the supporting soil, the outer wells of these caissons were stopped and decked over at elevation -40, and all the

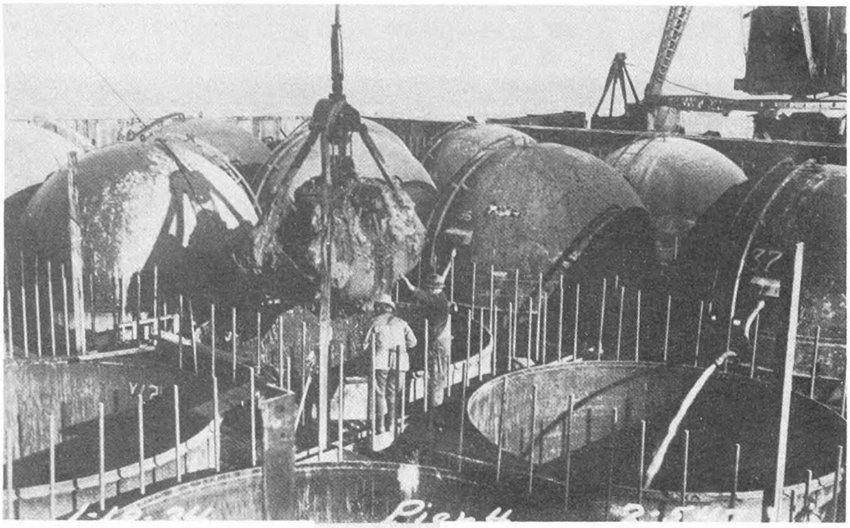


Fig. 15

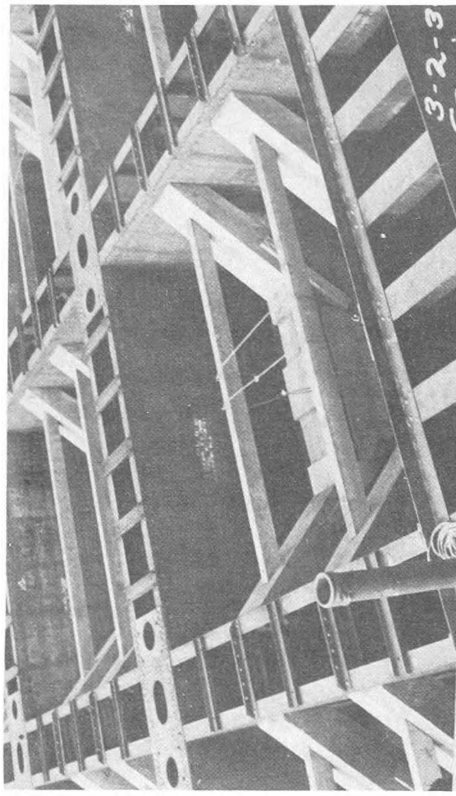


Fig. 22

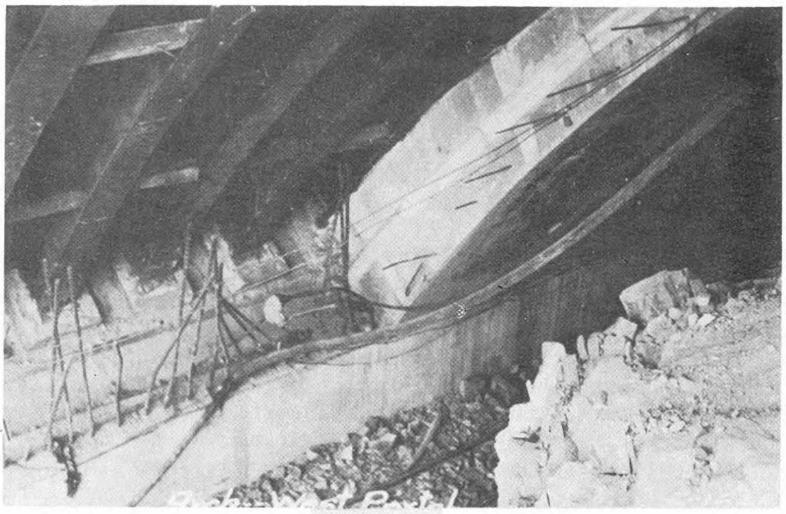


Fig. 25

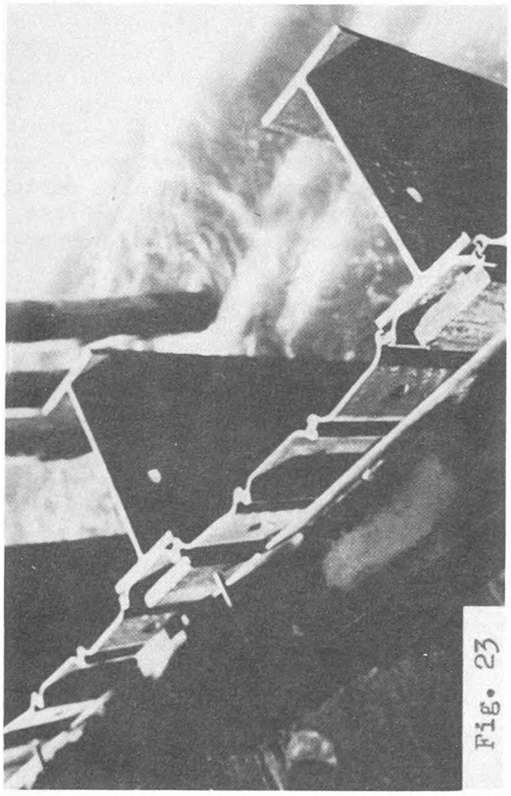


Fig. 23

Fig. 26

wells were left filled with water only.

The steel sheet pile cofferdams for the pile supported piers were braced by master piles of 36 inch I beams welded to every fourth sheet pile. (Fig. 23) This method of installation has been used before, but not with the master piles on the outside of the sheeting. As the cofferdams were pumped out, the sheeting between master piles deflected enough to tighten the joints and only a small amount of leakage occurred.

Tunnel Construction. The tunnel through Yerba Buena Island, connecting the West Bay suspension structure and the East Bay cantilever structure, is of interest because of its great diameter and the method of its construction. While the tunnel is only 540 feet in length, it has the world's record excavation width of 79 feet at the spring line. It provides the same double deck roadway facilities as the bridge, including a reinforced concrete floor for the upper six lane roadway, and a lower deck with a truck roadway 31 feet wide x 16 feet in height and a 27 ft wide x 20 ft high double track interurban railway.

Before the tunnel was designed, the rock formation of the Island was thoroughly explored by core drilling and the geology carefully studied. The interbedded sandstones and shales, forming the Island, have been subjected to severe movements which have shattered the rock, making a tunnel installation of this width both difficult and hazardous. To consolidate the roof rock as much as possible, horizontal grout holes were drilled from the west portal, a distance of 200 feet, and grouted under a pressure of 100# per square inch.

The construction of the tunnel involved six (6) successive operations (Fig. 24):

(1) An 8 ft x 8 ft pilot tunnel was drifted through at the crown, while two 14 ft x 14 ft pilot tunnels were drifted through at the bottom of each side wall.

(2) Additional side wall drifts were shot down from the roof until space for the side wall constructions had been prepared.

(3) The concrete side walls were built.

(4) The roof was excavated in sections and the muck from such operations dropped into the side wall drifts between the core and the inner face of the concreted side walls. Each section of the roof excavation was closely followed by the installation of the roof bracing and struts.

(5) The roof arch was concreted.

(6) The rock core was removed.

Steel ribs of 16 inch 78# H beams, bent to conform to the curve of the extrados, and spaced 3 feet on centers, were used as roof timbering, and pressed steel roof plates or poling boards were installed to prevent the fall of loose rock. After placing the intrados forms, all spaces above such forms were filled with concrete in the installation of the roof arch. The roof ribs are supported on the previously constructed side walls. Immediately following the installation of the arch roof ribs, and before the excavation of the rock core, the reinforced concrete roof arch is placed. (Fig. 25)

Conclusion. The bridge construction is now so far progressed that arrangements have been made for its opening on November 7th of this year, the date of the annual football classic between Stanford and California University. (Fig. 26) The Pacific Fleet of the United States Navy will be anchored in San Francisco Bay to contribute to the grand festivities planned for this occasion. In the spring of 1939, when the Golden Gate Bridge also will have been opened to traffic, the San Francisco World's Fair will be inaugurated in honor of these two record breaking structures.

No. N-11

FOUNDATION OF MODERN BRIDGES IN DENMARK

Aage E. Bretting, Chief Engineer, Christiani & Nielsen, Copenhagen, Denmark

Denmark consists of the peninsula of Jutland, which is cut through by the Limfiord, the main islands of Zealand, Funen, Lolland and Falster, and some three hundred smaller islands. The many Fiords and Belts which split up the country were felt to be a great hindrance to traffic development, and efforts have recently been made to connect the different parts of the country by bridges.

The most important of the bridges which have been carried out during the last years or are in the course of construction, are the following:

Combined bridge over the Little Belt between Jutland and Funen.

Combined bridge over the Sound Storstrommen between Zealand and Falster.

2 bridges for railway and highway respectively over the Limfiord between the towns of Aalborg and Norresundby.

Combined bridge over the Limfiord at Oddesund.

Combined bridge over the Alssund between the island of Als and South-Jutland.

Road bridge over the Guldborgsund between Lolland and Falster.

2 bridges for railway and highway respectively over the Roskilde Fiord at Frederikssund.

Plans for several bridges are ready and will probably be carried out during the next few years, viz.:

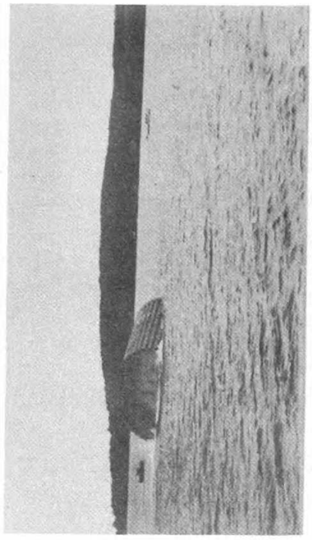


Fig. 9

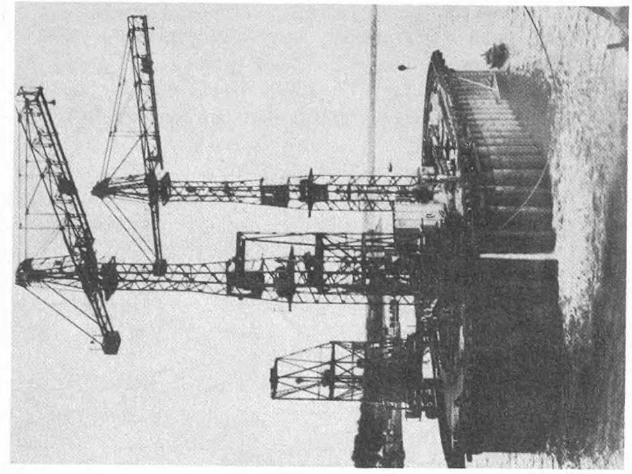


Fig. 11

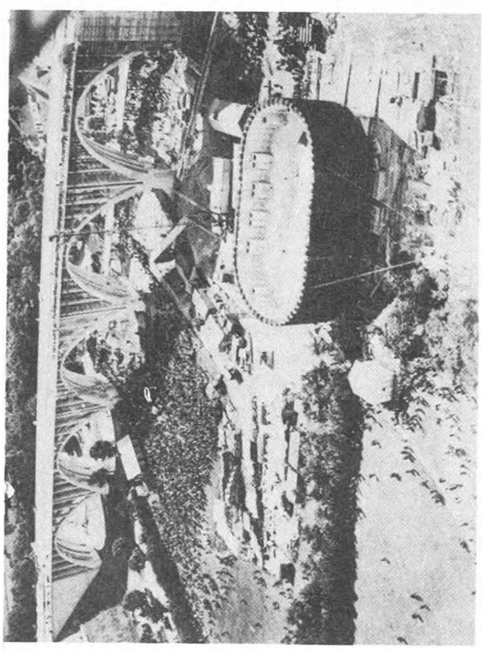
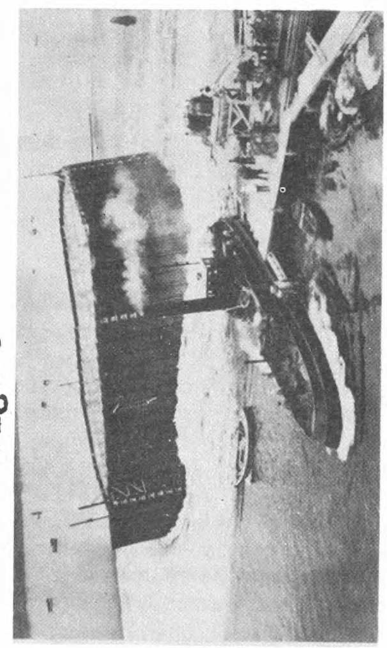


Fig. 6



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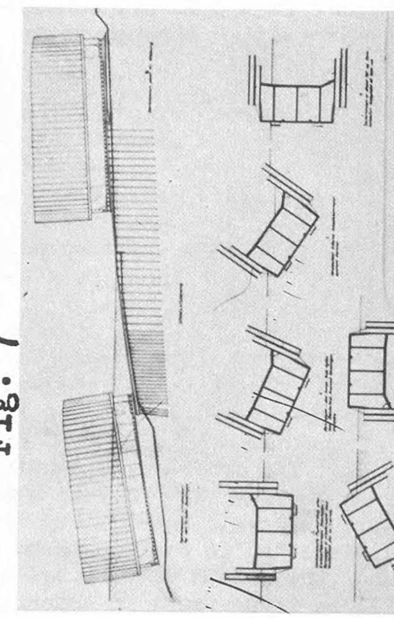

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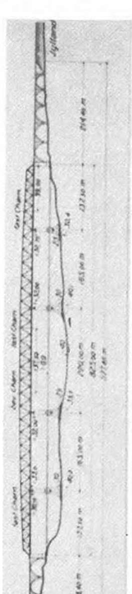
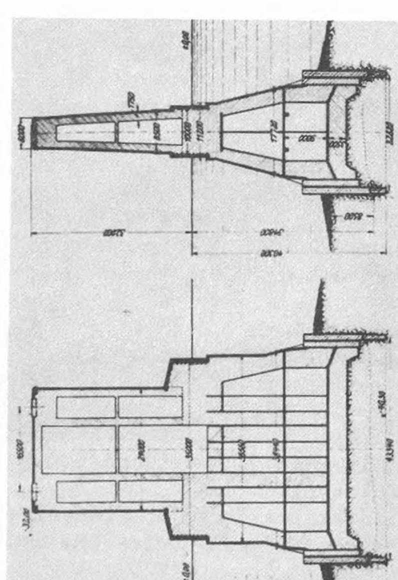


Fig. 3



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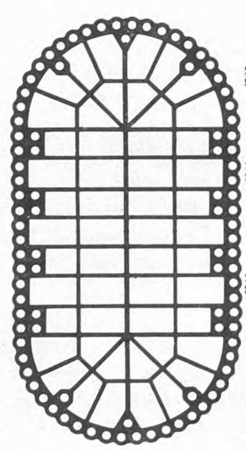


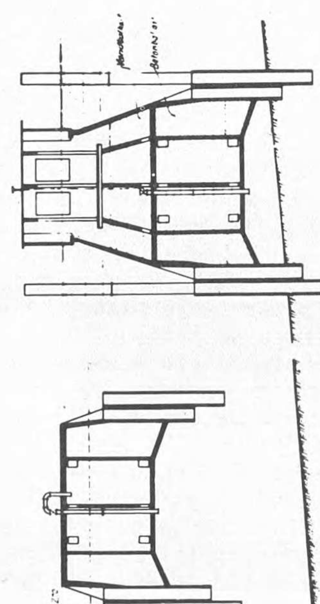


Fig. 5

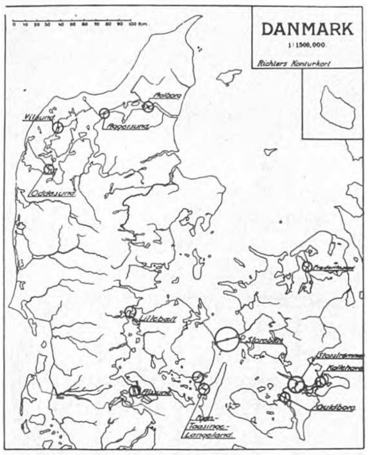


Fig. 1

Road bridge at Vilsund connecting the island of Mors with North-Jutland.

Road bridge at Aggersund over the Limfiord.

Road bridge from Funen via Taasinge to Langeland.

Road bridge at Kallehave between Zealand and Moen.

Finally the plans for the two large bridges over the Sound between Copenhagen, Denmark, and Malmo, Sweden, and over the Great Belt have been thoroughly studied.

The surface of Denmark is generally covered by boulder clay. Only in a few places tertiary clays and chalks will be found. In the numerous fiords and sounds the bottom is often covered by extensive layers of mud and silt. Pile foundations have been extensively used, and at a number of modern bridges a special Danish type of foundation, which has proved to be very effective and economical, has been employed. This method is devised by Professor Engelund. (See Paper N-4, Vol I) Great care is taken to design the pile foundations in such a way that vertical as well as horizontal forces can be transferred safely to the bottom, and the greater part of the piles is usually driven on a batter. Very great lengths of piles of wood and of reinforced concrete have been used (up to 36 m long). Danish Engineers have also developed rational methods for statical calculations of load distribution on the piles.

The most important examples of bridge foundations in Denmark, the bridge over the Little Belt and the bridge over Storstrommen, have, however, been founded directly on the clay. At these bridges novel methods of foundation have been employed, and a description of these methods will be given. Further, as an example of bridge foundation on piles, the bridges over the Limfiord at Aalborg will be mentioned.

Bridge over the Little Belt. Fig. 2 is a cross-section of the bridge showing double track railway, a 5.6 m road and a 2.5 m footpath. On Fig. 3 is seen the general lay out of the bridge. There are five main spans of 137.5, 165, 220, 165, and 137.5 m and reinforced concrete arches on both shores. The clearance under the bridge is 33 m. The main girders are cantilever beams with 2 hinges in the main opening and one hinge in each of the shore spans. This steel superstructure is from an European point of view quite important, but the most interesting part of the bridge is no doubt the foundations. The depth of water reaches 40 m, the currents in the belt are strong, rather more than 3 m per sec, and also severe ice pressures are to be expected. The piers are built in water depths of about 30 m. The bottom consists of a very fine-grained homogenous tertiary clay of great resistance, down to unknown depths. The clay is practically impervious and is not weakened when in contact with water. Observations during several decades have shown that no changes to the bottom in the Belt have taken place during this time.

The bridge was designed by and carried out for the Danish State Railways. The special method of foundation designed by the contractors Monberg & Thorsen, Copenhagen, and Grün & Bilfinger, Mannheim, was based on the above-named characteristics of the clay.

Fig. 4 shows the finished pier consisting of a reinforced concrete caisson stiffened by lateral and longitudinal walls. At the bottom along the outer edge this caisson has a row of tubes of reinforced concrete lined with steel. These tubes, forming a complete wall around the caisson, are sunk down into the clay by the weight of the caisson and by boring out the clay inside the tubes. Inside the caisson water was pumped out, and the working chamber could be laid perfectly dry, without resort to compressed air. Fig. 5 shows the caisson in its first stage. The lengths of the tubes have been adjusted to the form of the seabed. The lower part of the caisson, consisting of the tubes, sides and two lower decks, was built on a slipway, bottom up. (Fig. 6) The weight of this part was about 6400 tons. Fig. 7 shows the launching of the caisson. Fig. 8 shows the next operation, turning the caisson over. It had a complete system of pipes and valves, so that water could be let into the different compartments and later pumped out again. At one side the tubes were filled with sand; the compartments on the same side were filled with water and finally valves connecting the working chamber with the sea were opened, thereafter the caisson turned over in the course of about half an hour. During the turning the sand ballast falls out of the tubes, and the caisson can be balanced afterwards by pumping in the compartments. A water depth of 30 m was needed for this operation. (See Fig. 9) At a suitable location the caisson was then placed on the sea bottom and the walls built higher. (See Fig. 10) This operation was repeated 3 times until it reached its final height and could be placed in position on the site of the pier.

The caisson was built as a closed box to a height corresponding to 3 m below water level in the pier. Above this level the surface was to be covered with granite, and the possibility of correcting the position of the pier shaft should be taken into consideration. The upper part, therefore, was made as a staging in reinforced concrete, supporting the necessary cranes, boring apparatus, etc., and also the upper cofferdam, inside which the pier was concreted.

The reinforced concrete tubes were connected to steel tubes of same inside diameter reaching above the water level, and inside these tubes the clay was bored out by means of special boring apparatus. The clay was cut so fine that it could be pumped up by compressed air.

From the working chamber up through the entire height of the pier there were placed pipes, through

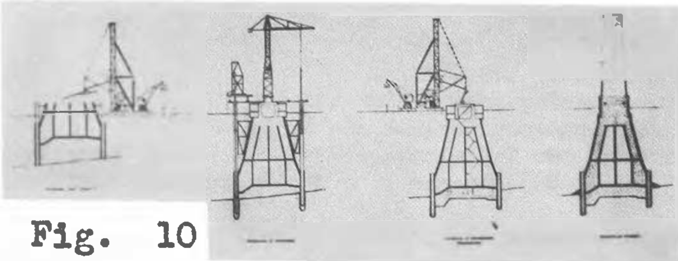


Fig. 10

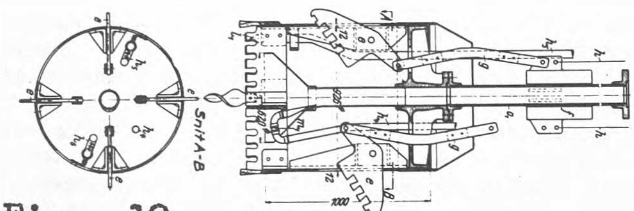


Fig. 12

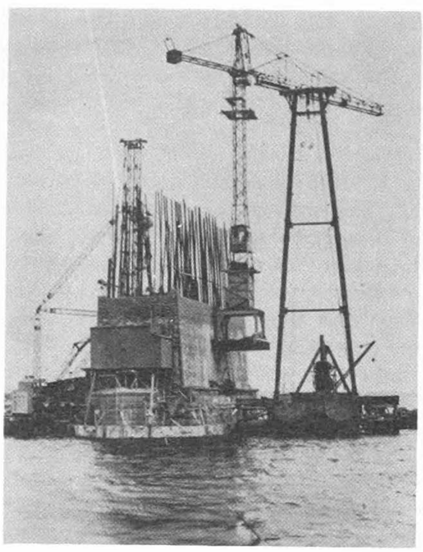
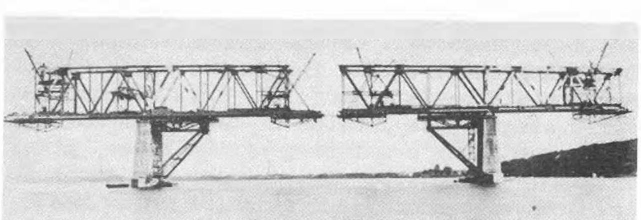


Fig. 13



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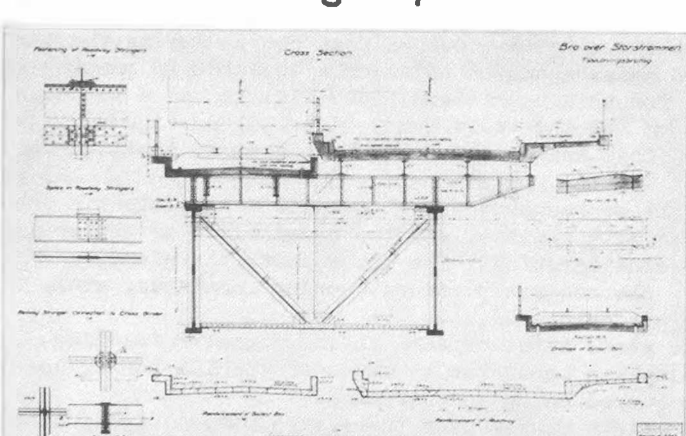

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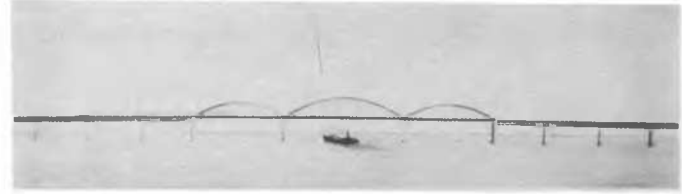

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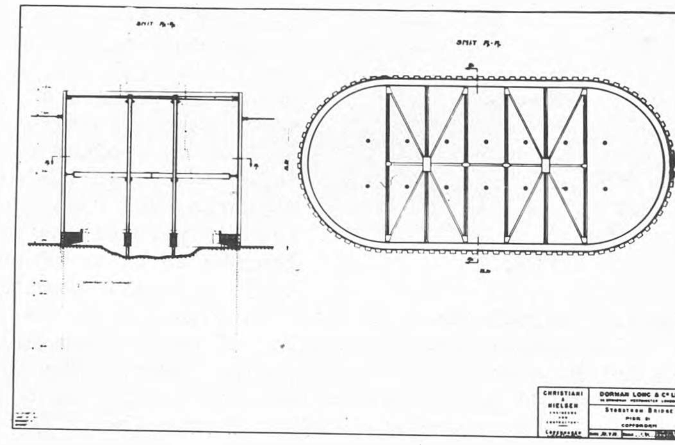

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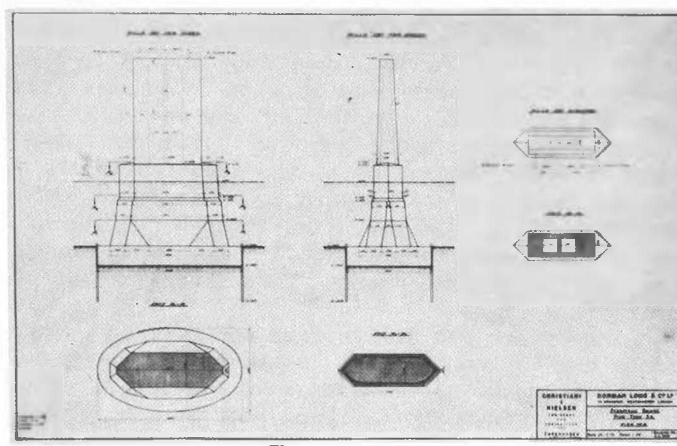


Fig. 19

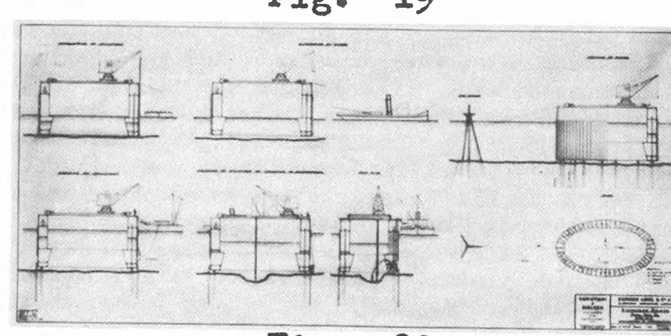


Fig. 20

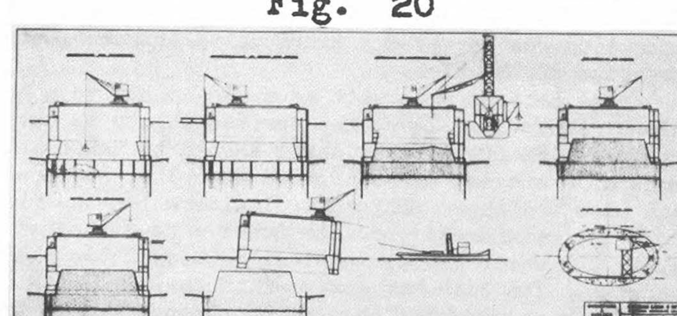


Fig. 21

which the water displaced during the sinking of the caisson, found an outlet. By regulating the water level in these stand-pipes the rate of sinking of the caisson was controlled. (During the turning of the caisson the air from the working chamber could escape through these pipes). Before the excavation in the working chamber commenced, the tubes along the outer edge were filled with concrete under water.

The caisson further sank a few meters during the pumping out and excavation, and to control the movements the excavation was carried out in sections, and the roof of the working chamber strutted against a concrete slab on the bottom of the excavation. In this way the edge line of tubes was sunk to about 10 m below sea bottom; the working chamber and the outside compartments of the pier shaft were then filled with concrete.

The interior compartments were left empty and are being held dry by automatically acting pumps, so as to reduce as far as possible the dead load on the foundation. Fig. 11 shows the steel tubes projecting above the water surface and the boring apparatus used for the excavation in the tubes. Fig. 12 gives details of the boring crown itself. Fig. 13 shows the construction of the pier shaft above water. Fig. 14 shows the method of erection. Steel brackets were built out from the piers and the superstructure erected by free cantilevering.

The total cost of the bridge proper was about 23 Mill. Kr. ($5,000,000).

The Storstrom Bridge. This structure is from a technical point of view not so difficult as the Little Belt Bridge, but the great length, about 3.2 km for the bridge proper, places it among the more important European bridges of recent years. A cross-section of the bridge is seen on Fig. 15. It accommodates a single-track railway, a road-way 5.6 m wide, and a foot-path of 2.5 m. The general lay out of the bridge is seen on Fig. 16. Three navigation openings of 104, 138, and 104 m are situated about in the middle of the bridge. The clearance below the bridge is at this place 26 m. The approach spans numbering as much as 47, consist of spans of alternatively 58 and 62 m length.

The main girders are plate girders of the deck-cantilever type. In the navigation openings the plate girders are reinforced by arches. The bridge thus contains 49 piers and 2 abutments. A longitudinal section in the bridge line is shown on Fig. 17.

The water is relatively shallow, maximum depth about 14 m, on an average 8 m. The bottom consists mainly of boulder clay of considerable firmness, near the shores covered by a layer of mud, a few meters thick. A detailed account of the soil studies made for this bridge is given in the author's paper "Soil Studies for the Storstrom Bridge " (Paper No. Z-5, Vol I). The great number of the piers and the relatively uniform conditions of water depth and soil quality made it possible to standardize the construction of the greater part of the piers.

The four piers at the navigation openings, which were considerably larger than the normal ones, and a few piers near the north shore were, however, built in the ordinary way by means of a steel sheet piling cofferdam carried above the water surface. On Fig. 18 this cofferdam is seen. The water depth is 9 to 10 m. Only bracings at 2 levels are used, one lightly constructed frame above water and one heavy steel frame about 6.5 m below water. With this bracing the pit was laid dry, and a heavy frame of reinforced concrete was built directly on the bottom. After this frame had hardened, the excavation was carried out, and the foundation slab, and the pier shaft was concreted. The steel sheet piling was afterwards cut off below water along the upper edge of the foundation slab.

Twenty-seven of the normal piers resting on firm clay bottom, were built by means of two so-called Universal-Units of different size. The type of these piers is shown on Fig. 19. The foundation slab is elliptical in plan, generally about 3 m thick. The lower part of the pier shaft is formed so that the slab projects not more than about 2 m. The part of the shaft which is near the water level, is covered with granite ashlar. The upper part of the pier shaft is hollow and at the top covered with a reinforced concrete beam. The use of the Universal-Unit is illustrated by Fig. 20 & 21.

The Unit built in steel is formed as a ring-shaped pontoon whose outside dimensions correspond to those of the foundation slab. The interior face of the lower part is formed according to the pier shaft and serves as a form for this shaft. The Unit floats with a draught of about 3 meters. Internally it is divided by bulkheads into different tanks which can be filled with water or emptied by electrically driven pumps. At the deck there is an electric crane serving the whole area covered by the Unit.

The steel sheet piling is placed in position along the external circumference, all piles locked together and hung on the Unit. On the site of the pier a line of wooden piles has been driven in advance and cut at the same level as the upper side of the foundation slab. The Unit is lowered on these piles, and the steel sheet piling is driven under water by means of a Mac Kiernan Terry hammer, till the upper end is just above the lower edge of the Unit. At this place the Unit has a bulge shaped projection on the exterior face with a sloping upper surface. In the upper part of the sheet piling alternate grooves have been filled with wood, so as to obtain a smooth surface on the inside. In the wedge-shaped joint between the piling and the bulge a continuous hemp rope, boiled in tallow, is placed, and water is pumped out inside the cofferdam. The joint tightens itself, a diver helping, if necessary. The bed is laid dry and excavation of the foundation can take place.

No bracing at all was employed. The Unit itself was designed to take the entire water pressure on its outside and also the reaction from the top of the steel sheet piling. The lower end of the piling was solely supported on the clay. When the excavation was finished the foundation slab was concreted against the steel sheet piling, which remained in place as a protection against scour. The lower part of the pier shaft was then concreted, partially using the Unit as a form. The work was stopped when the pier shaft had reached 3 m below water level.

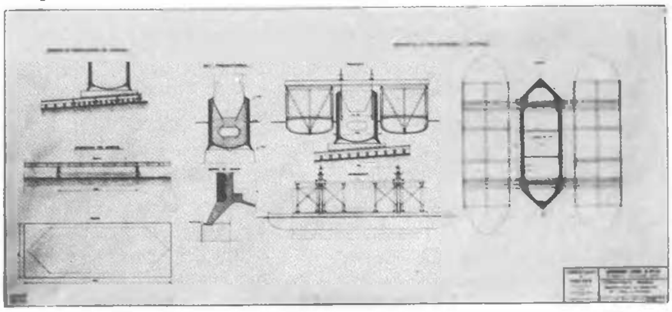
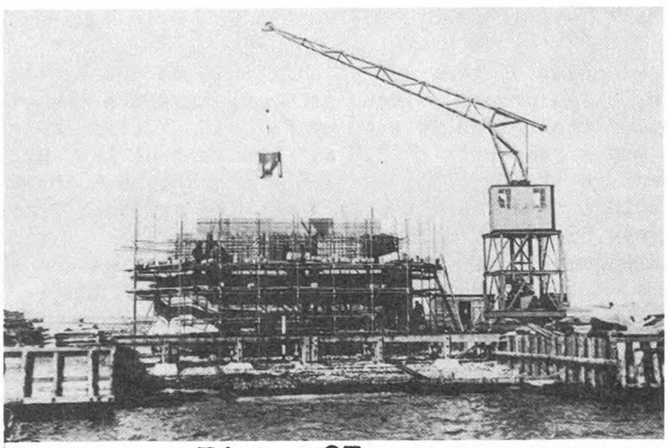


Fig. 22



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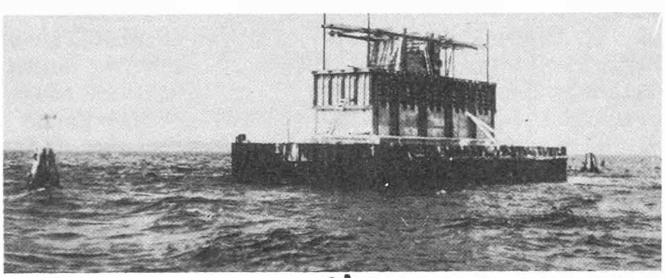
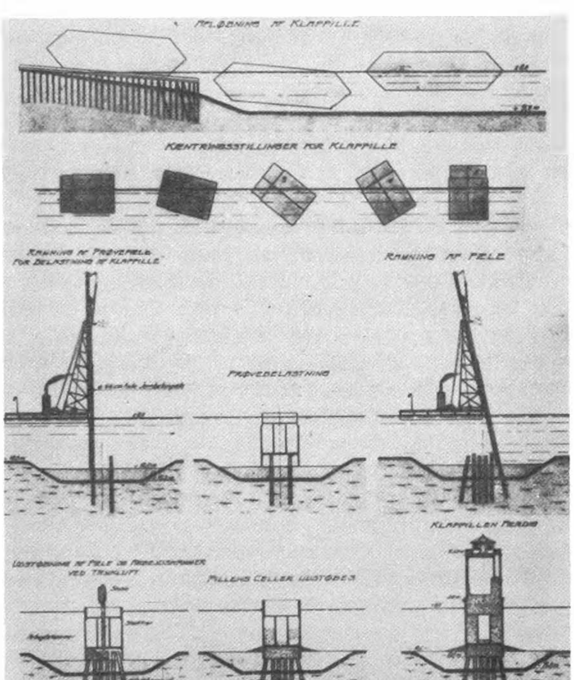


Fig. 24



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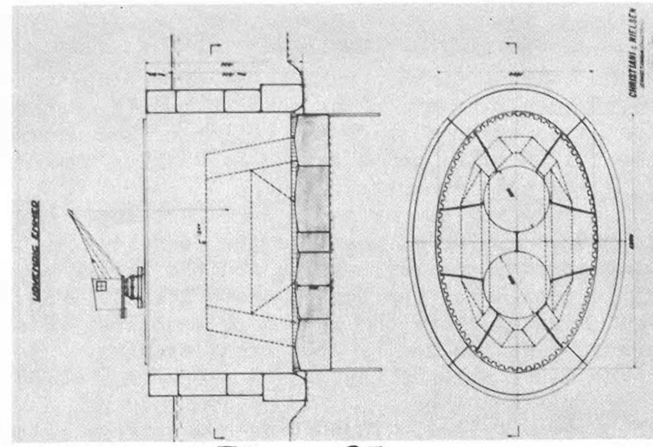

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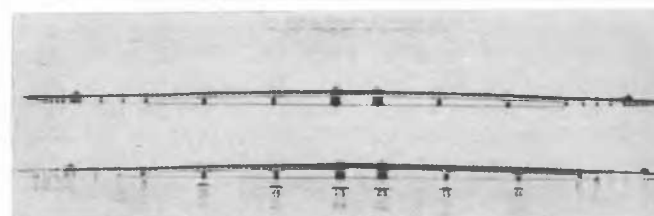

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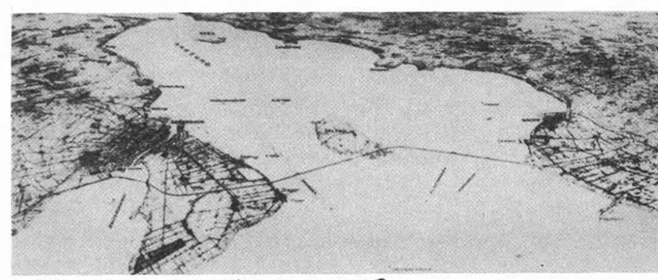


Fig. 32

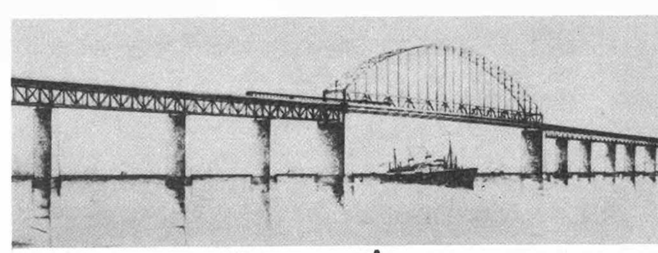


Fig. 34

Fig. 35

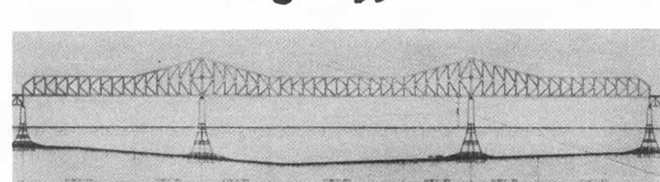


Fig. 37

When the concrete had hardened, the Unit was removed. It was jacked loose from the pier by means of wooden struts placed in vertical tubes in the tanks of the Unit. These struts rested on the surface of the foundation slab, and between the upper end of the struts and the deck of the Unit screw jacks were employed. Further the buoyancy of the Unit could be used at will.

The part of the pier shaft which was between 3 m below water and 2.5 m above, was constructed on a slipway as a reinforced concrete caisson without bottom, covered with granite ashlar on the outside. (Fig. 22 and 23) The carriage with the completed caisson was lowered into the water and the caisson, weighing about 250 tons, was suspended between two barges and towed to the pier site. Here it was lowered in exact position by means of guiding tubes fitting in holes in the top of the pier. Previously an asphaltic tightening substance had been placed on the top of the pier at elevation -3 m, so that the water could immediately be pumped out, and the caisson filled with concrete.

All these operations, from the time when the Unit was placed in position on the pier site until the pier was finished up to elevation + 2.5 m, were generally carried out in less than a month. The upper part of the pier shaft was concreted in the ordinary way inside steel forms. (Fig. 24) The piers built in this manner were founded at depths of 8 - 12 m below sea level. Fourteen of the piers were founded at greater depths up to 16 m below sea level. The bottom at these piers proved less resistant, and as the water pressure was greater, the aforementioned method was modified so that the foundation slab was concreted under water before the pit was laid dry. They were built by means of 2 "external" Units of equal size. Fig. 25 shows these Units forming a cofferdam for the pier shaft.

The steel sheet piling is driven along the inside circumference. Excavation is made under water by a grab, and the foundation divided into 10 sections by means of 2 steel cylinders in the centre and radial walls of reinforced concrete. Each section was concreted separately through a vertically placed tube. When the foundation slab was completed, the wedge-shaped space between the steel sheet piling and the sloping inside of the Unit was filled with concrete also under water, and when all the concrete had hardened sufficiently, the cofferdam could be pumped out. The thickness of the foundation slab was generally about one-fourth of the foundation depth. The pier shaft was built inside ordinary wooden forms up to elevation -3 m, whereupon the Unit was removed and the pier completed as mentioned before.

As to the erection of the steel work it is only to be mentioned that the complete steel spans were placed on the top of the piers by means of a powerful floating crane of 500 tons capacity. Fig. 26 shows this crane and the present state of the works.

The bridge was designed by and constructed for the Danish State Railways. The superstructure was carried out by Dorman, Long & Co., Middlesbrough, England. The substructure by Christiani & Nielsen, Copenhagen, Denmark, who have evolved the special methods of foundation which were employed.

The cost of the Storstrom Bridge proper will be about 28 Mill. Kr. ($6,000,000). Including approach works, etc., about 40 Mill. Kr. ($9,000,000).

Bridge over the Limfiord at Aalborg. The bridge has an 8.5 m roadway and 2 footpaths of 3.0 m each. The general lay-out of the bridge is seen on Fig. 27. In the middle is situated a bascule bridge of 30 m free width, on each side of which there are 3 openings of 56, 67 and 56 m respectively. The superstructure is steel plate-girders of the deck-cantilever type.

By far the most interesting part of this bridge is the foundation, as the bottom conditions were rather difficult. The water depth is only about 10 m, but the upper part of the bed in a depth of about 17 m consists of mud with insignificant carrying capacity. Not until a depth of about 30 m below sea level, are strata of sufficient carrying capacity found, generally consisting of rather fine sand. The piers rest on piles of 25 - 35 m length, driven 5 to 6 m into firm ground, the foot of the piles generally being about 35 m below sea level. The type of pier is shown in Fig. 28. (This figure is identical with Fig. 1 of Paper No. N-4, Vol I and is not reproduced here).

The heads of the piles are about 10 m below sea level, corresponding approximately to the water depth, and above this level the piers are solid.

The piers were built in the following manner: A reinforced concrete caisson was built lying sideways on a slipway, launched and covered with granite ashlar while floating. (Fig. 29) First, at the site of the pier the mud was excavated by a ladder-dredger to elevation -15 m. Sand was next filled in by pumping to elevation - 12.0 m.

The piles were hollow reinforced concrete piles of 66 cm external diameter. The thickness of the walls is 8 cm. Fig. 30 shows details of these piles (it is identical with Fig. 6 of Paper No. N-4, Vol I and is not reproduced here.) Four vertical piles were driven, one at each corner of the pier, with their heads at elevation -9 m. (Fig. 29). These piles were test-loaded with a load of 200 tons on each pile. The load of 800 tons was supplied by placing the large caisson for one of the bascule piers on the 4 piles and filling with water. The load remained on the four piles for a duration of not less than 14 days. During this period no settlements were observed.

The other piles were then driven, all as batter piles with an inclination of 1:5, their heads a little lower than the vertical ones. There were 60 - 90 piles in one pier. The caisson was then placed in the right position on the 4 vertical piles, ballasted by water and concrete, and sand and stone dumped around the caisson. The working chamber was laid dry by compressed air. A bottom layer of concrete 1 m thick, resting on the sand cushion was poured, and the rest of the work could be carried out without compressed air. The top of the hollow piles was cut away so that they could be filled with concrete. The working chamber was filled up with concrete, whereafter the different compartments of the caisson could be filled, and the pier completed. Fig. 31 shows the heavy floating pile-

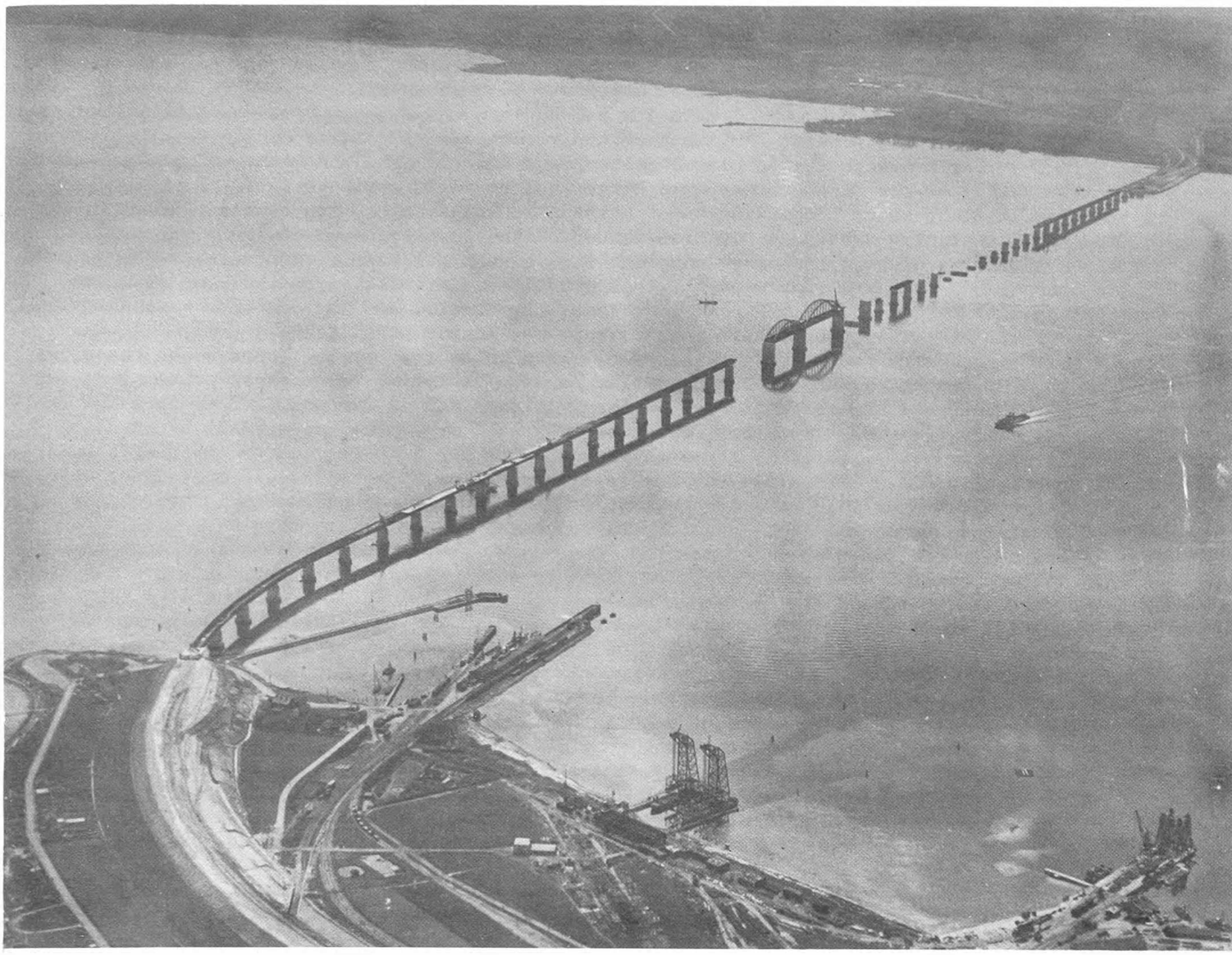


Fig. 26

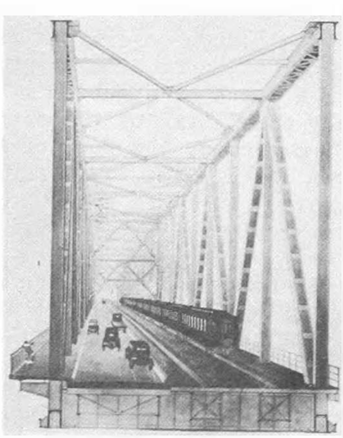


Fig. 2

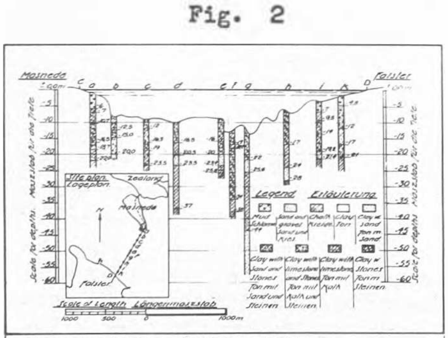


Fig. 17

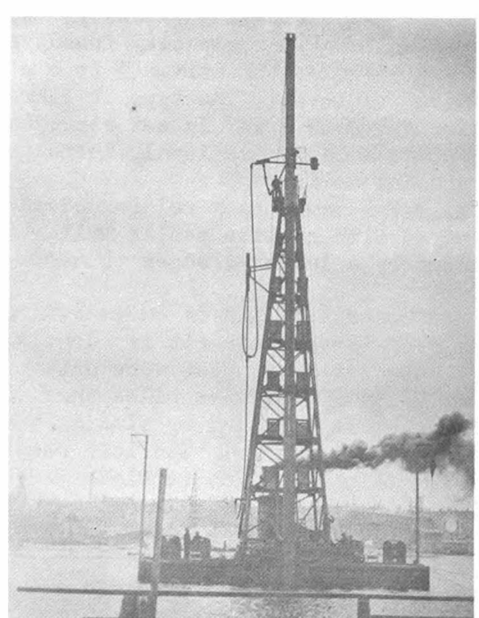
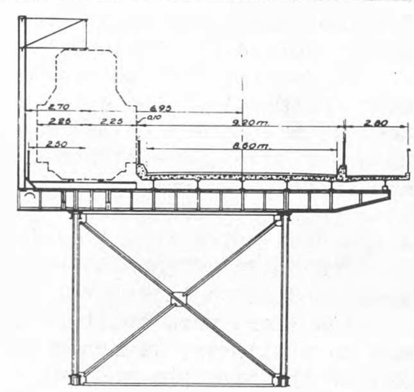


Fig. 31

Fig. 33 (right)



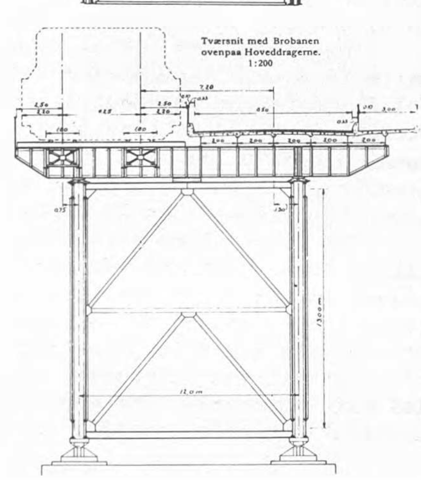


Fig. 36 (right)

driver used for the driving of the piles. The weight of the hammer is 6 tons. The work was designed and carried out for the Town of Aalborg by Kampmann, Kierulff & Saxild, Copenhagen.

Finally a few pictures of the proposed bridges across the Sound and the Great Belt will perhaps be of interest. The plans for these bridges were recently published by the three Danish engineering firms Christiani & Nielsen, Hojgaard & Schultz, and Kampmann, Kierulff & Saxild.

The Bridge across the Sound. It connects Copenhagen, Denmark, with Malmö, Sweden, via the islands of Amager and Saltholm, as shown in the aerial view, Fig. 32. The total length of the three stretches of the bridge proper is about 15 km. The bridge accommodates a single track railway, a roadway 9 m wide and a path for pedestrians and cyclists, as shown in the cross-section, Fig. 33. There are 2 main openings at Drogden and Flintrännan, each a 300 m span with 45 m clearance (Fig. 34), one span of 200 m at Trindelrännan and a great number of spans of about 70 m and smaller.

The water depth does not exceed 11 m; the bottom mainly consists of chalk, and the foundation conditions are relatively simple. Similar methods of construction as employed for the piers of the Storström Bridge are intended.

The cost of this bridge has been estimated at about 150 Mill. Kroner ($34,000,000).

The Bridge across the Great Belt. It connects Zealand and Funen via the island of Sprogö as shown in the aerial view (Fig. 35) has a length of the bridge proper of about 15 km. It accommodates a double track railway, a roadway of 9 m width and a path for pedestrians and cyclists (cross-section Fig. 36). In the East Channel is one main span of 400 m with a clearance of 45 m and two side spans of 267 m (Fig. 37). At this place the maximum depth of water is 58 m, and the largest piers are to be built in 45 m of water. The bottom consists of clay and special methods of foundation have been evolved to meet these rather exceptional conditions of foundation.

This bridge has been estimated at about 260 Mill. Kroner ($58,000,000).

No. N-12 EXPERIENCES IN THE CONSTRUCTION AND WORKING OF MERCANTILE HARBOURS AT BREMEN AND BREMERHAVEN, AND THEIR VALUE IN THE DESIGN OF NEW STRUCTURES

Prof. Dr. Agatz, Harbour Engineer, Technische Hochschule, Berlin, Germany

General. Of the structures which I have designed and built during the past 18 years, or of which I have technical knowledge, I am only dealing with those marine structures which have for the most part had to be founded on a weak, disturbed subsoil with cohesive properties. It is the execution and superintending of such contracts which give us Engineers the best standpoint from which to make decisions for the planning of new structures.

The first contract involved the construction of a retaining wall (Fig. 1) about 1000 m long on timber piles, for the extension of Harbour No. 2 in Bremen during the years 1924/25. While sandy and gravelly subsoil was encountered in the first section, in the second half the Lauenburg clay encroached more and more into the region of the foundation. The slope was built over to provide as light a loading as possible for the clay and to reduce the lateral thrust on the structure. (Fig. 2) The danger of a foundation failure was avoided by driving the timber piles into the clay for a depth of up to 10 metres.

The second contract was for strengthening the Columbus Quay (Fig. 3)--the Customs Quay for large liners in Bremerhaven. Here very good results have been obtained during 30 years with small quays of timber raking piles. These structures had moved up to 26 cm during execution of the work, and in the first year of use owing to the cohesive subsoil and to the rather weak layer of sand, but the damage had not been of such an extent as to warrant consideration of the safety of the structures. Notwithstanding the greater height of the Columbus quay the same form of construction (Fig. 4) was retained and the wall founded on timber piles driven through soft silt (alluvial clay) into sand. The wall in the course of construction and subsequent use showed signs of movement--at first slow, then gradually increasing, and finally leading to a failure of the structure. After detailed investigations the structure was strengthened with a wall of steel sheet piling at the front and an anchorage at the back, (Fig. 5) and now in spite of having been twice deepened by 2 metres has withstood all demands imposed upon it.

The reason for the failure of Columbus Quay has been considered from the various points of view without hitherto determining the real cause. Although calculations of earth pressures by various experts have shown that the factor of safety does not quite reach 1, it cannot be concluded that ground failure has occurred. With only a very small increase in the angle of internal friction which owing to the nature of this ground is without doubt a possibility, a sufficiently high factor of safety could be obtained. The following are the reasons for the failure of the wall and for the considerable movements of 36 cm in the second section:

The movement of the second section (Fig. 6) has no real connection with the break in the first part. Here the movement is caused by the presence of clay in the foundation of the structure. It ceases with the rising of the as yet undredged river bed of the Weser before the last part of the

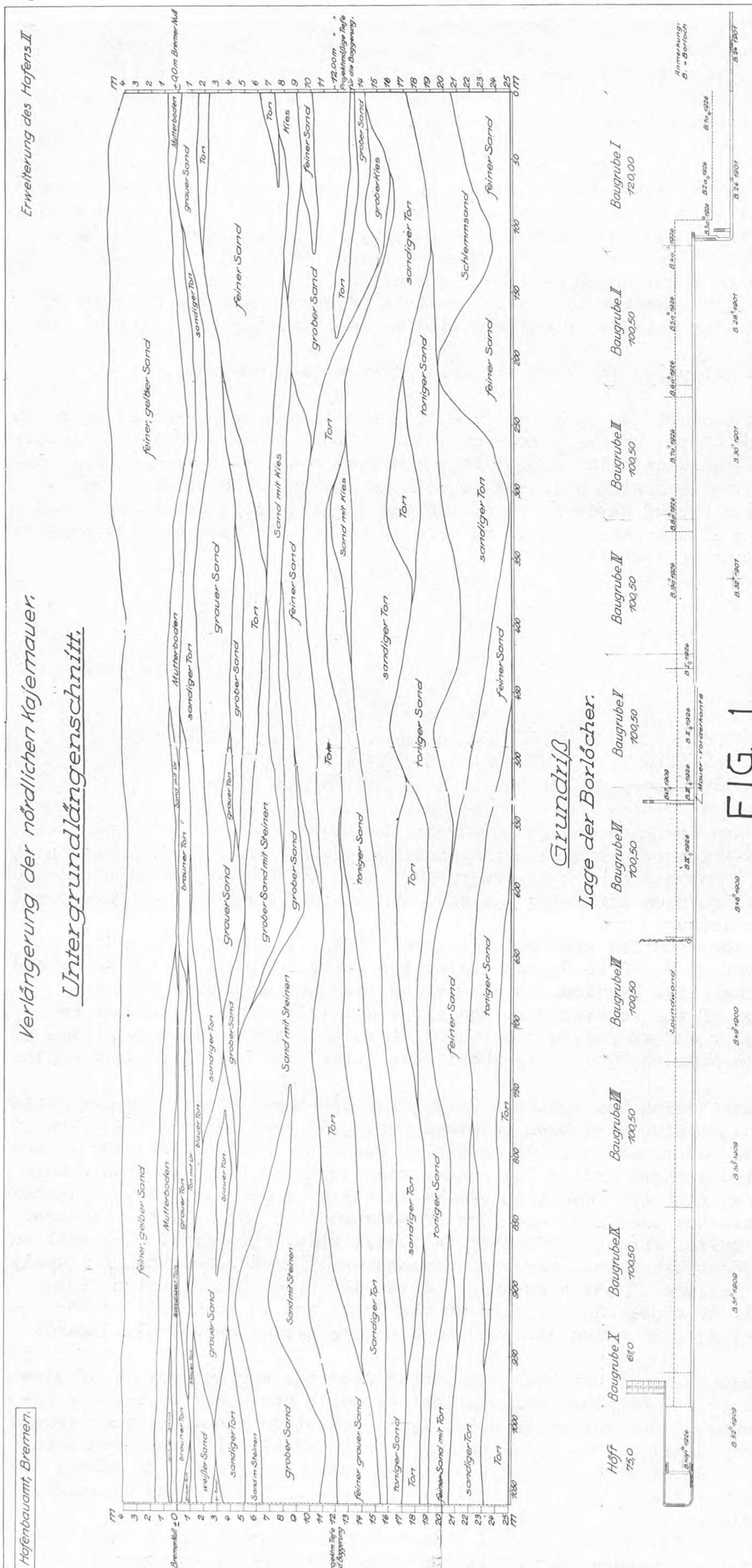
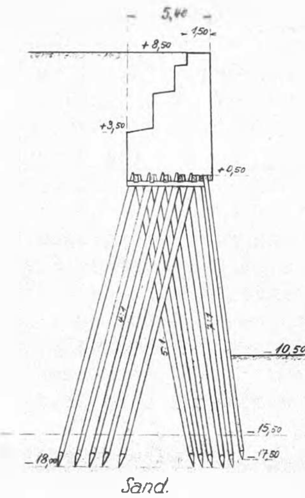
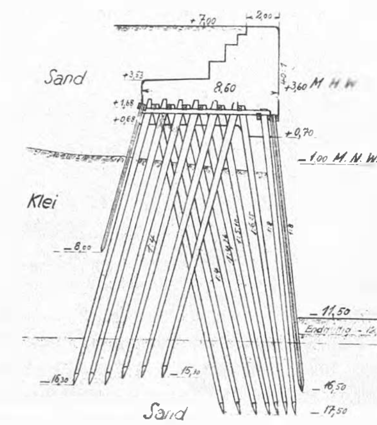


FIG. 1

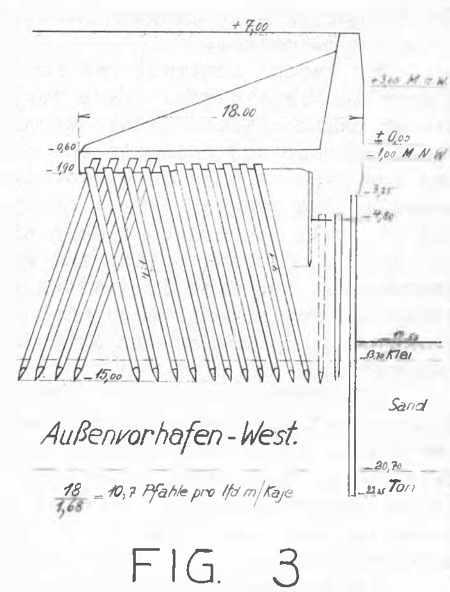


1. Kajenentwurf
Nordschleuse.



Columbusmauer.

$\frac{15 \text{ Pfähle}}{1,20 \text{ m}}$ = 12,5 Pfähle pro lfd. m/Kaje



Außenvorhafen - West.

$\frac{18}{1,68}$ = 10,7 Pfähle pro lfd. m/Kaje

FIG. 3

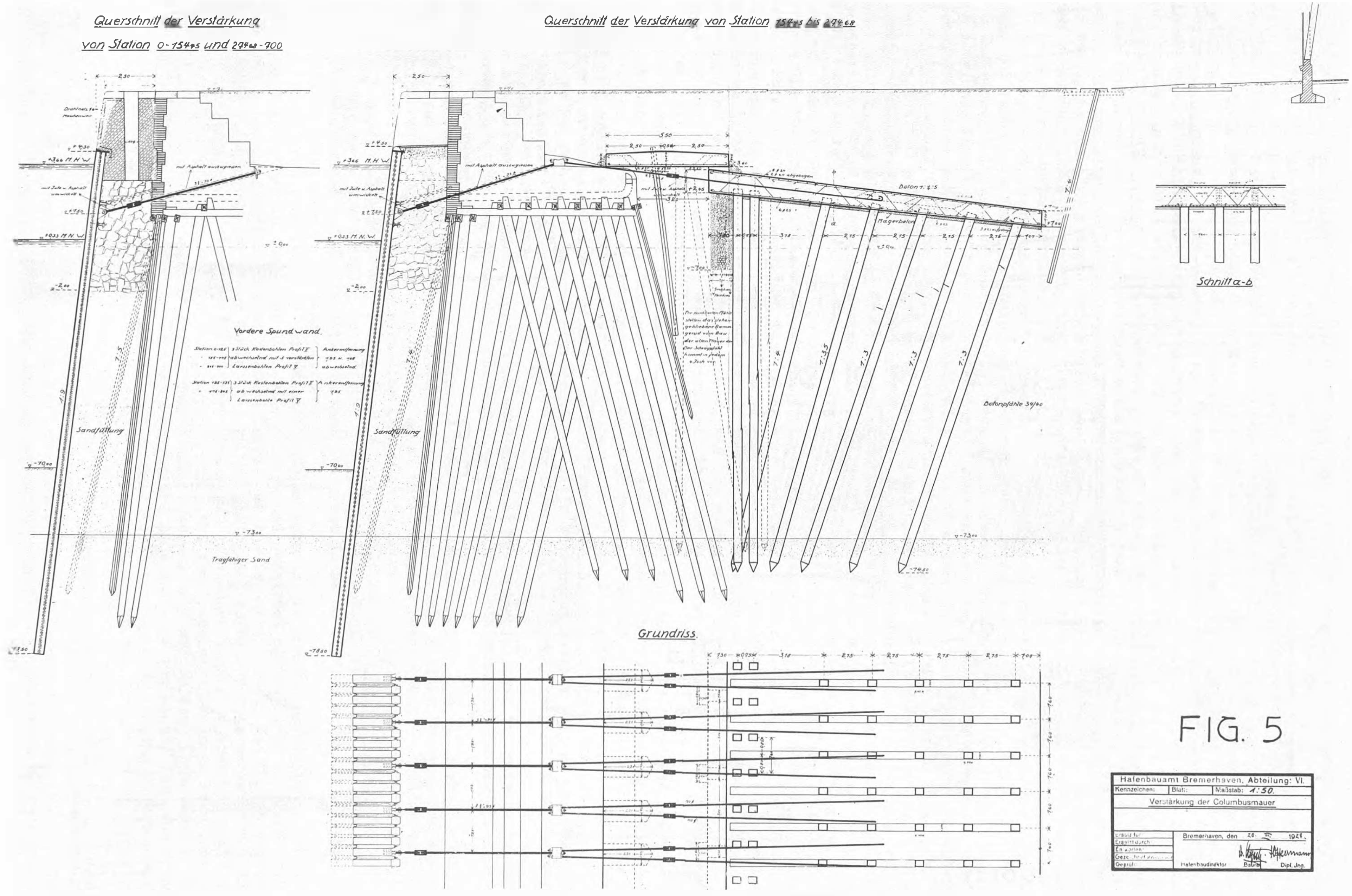

Querschnitt der Verstärkung von Station 0-154+5 und 274+68-700
Querschnitt der Verstärkung von Station 154+5 bis 274+68
Schnitt a-b
Grundriss
Vordere Spundwand
Sandfüllung
Tragfähiger Sand
Beton 1:4:5
Magerbeton
Betonpfähle 34/40
mit Asphalt ausgegossen
Hafenbauamt Bremerhaven, Abteilung: VI.
Kennzeichen:
Blatt:
Maßstab: 1:50.
Verstärkung der Columbusmauer
Bremerhaven, den
Hafenbaudirektor
Baurat
Dipl. Ing.

FIG. 5

N-12

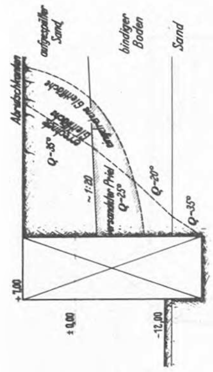
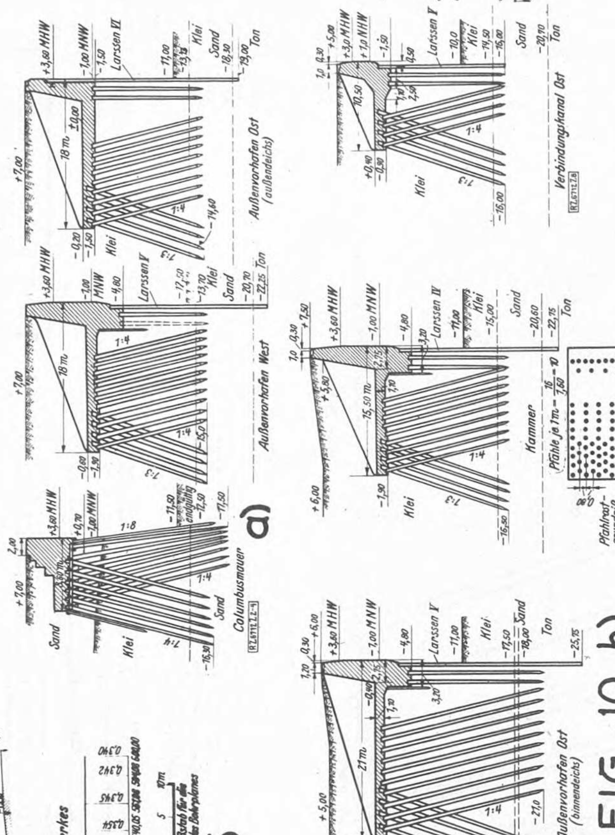


FIG. 7 (right)



a)

FIG. 10 b)

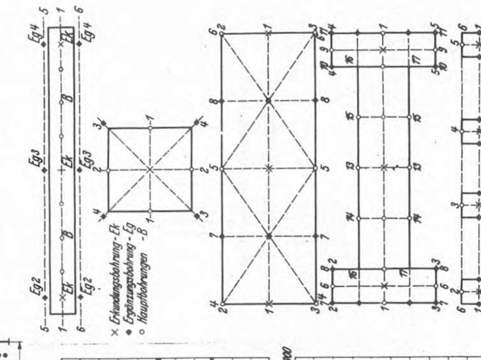
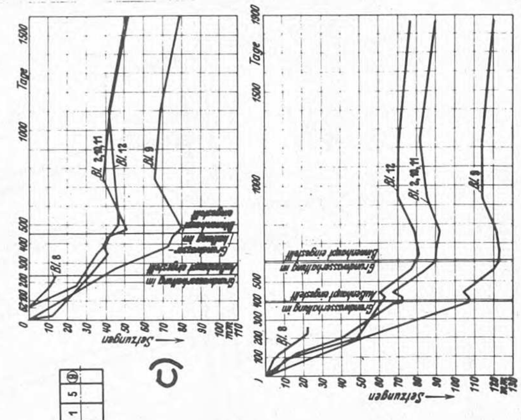


FIG. 19



a) c) b)

FIG. 13

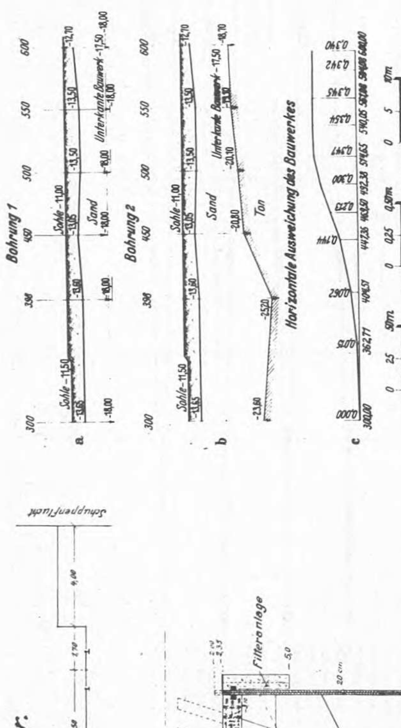


FIG. 6

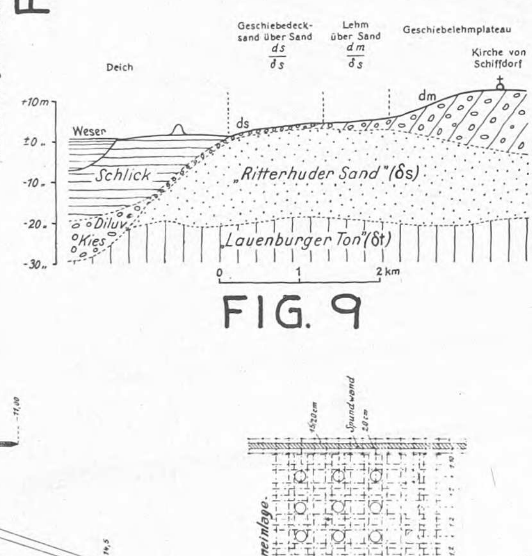


FIG. 9

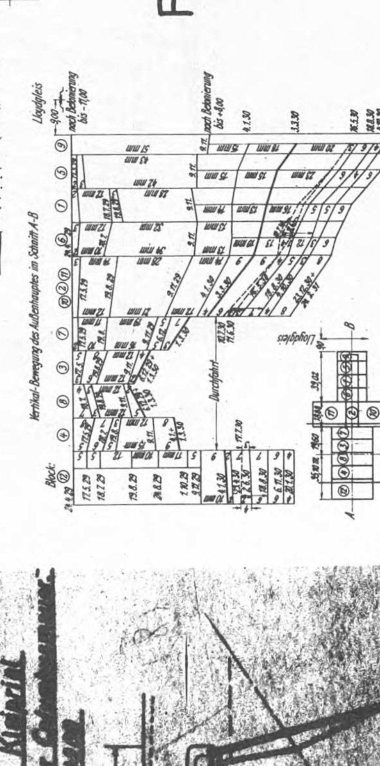


FIG. 12

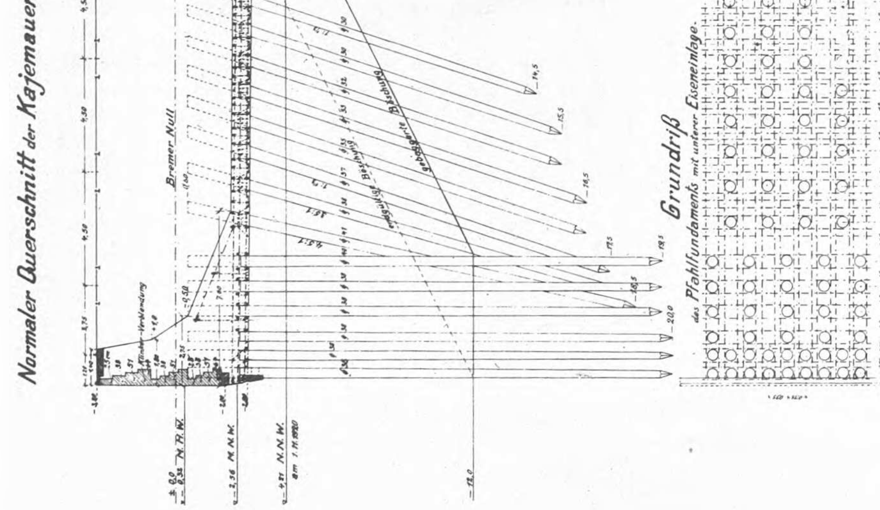


FIG. 2

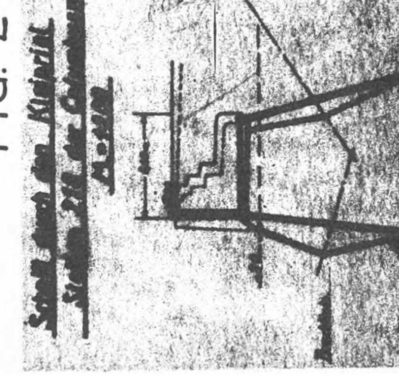


FIG. 8

contract. The fact that the wall failed here is due to the increase in subsoil water pressure as the drainage channels were silted up after a short time.

In the first section the thickness of the sand stratum under the structure was 10 to 12 m. The clay, therefore, had no effect here. On the other hand the wall crossed an old drainage channel at a very sharp angle (Fig. 7) so that instead of the theoretically flat plane of repose a much larger geological wedge came into consideration. Following this were: an increase in water pressure owing to silting of the drainage channel; considerable driving damage to the timber piles; large deflection, up to breaking point, of the tension piles in the reinforced concrete platform, caused by the compression of up to 1 m thick clay strata resulting from the subsequently deposited 9 m sand layer. Failure thus occurred in the wall simultaneously at the back and also at the front in the sheet piling. (Fig. 8) The piles located in between were unable to stand the increased stresses. The timber sheeting was fractured owing to the increased water pressure caused by the silting up of the joints between sheets. The resultant deflection caused a space to form between the sheeting and the clay, and the sand overlying the clay slipped into this space so that the sheeting was subjected to both earth and water pressure. It is noteworthy that the sheeting fractured in those places where it was known to have been damaged during driving.

The third section of the works involves the harbour extensions at Bremerhaven, and from these namely the North Lock Quays extension of Dry Dock, Swing Bridge and auxiliary works.

The geological foundation (Fig. 9) showed, as in Bremen, that in places the level of the Lauenburg clay rose into the region of the structure. Over the sand layer, which at times was only 1 metre thick, there was a layer of alluvial clay up to 15 metres thick. From information obtained in the construction of Columbus Quay, and measurements of deflections in neighbouring buildings, it was clear that the new works would have to be designed and constructed with the greatest care. The first essential was to limit slips and settlements to an amount which would not affect satisfactory operation. In the case of the quays the old form of timber pile construction was abandoned for a system dependent upon tension and compression piles. (Fig. 10) The standard Bremen practice could not be adopted owing to the time and expense involved. Neither could one build mass concrete walls owing to the load these would impose upon the clay, and consequent danger of ground failure.

Driving the timber piles without damage presented a considerable problem, and the piles were therefore spaced further apart and less importance attached to large penetration. The sheet piling served the purpose of taking up the horizontal earth and water pressures, and in order to construct this part as lightly as possible a relieving platform was provided at the top of the sheeting. The free length of the sheeting was thus reduced, and the sections of steel piling available on the market at the time could therefore be utilized. At the same time the danger of overloading the weak subsoil was eliminated by widening the piled foundation.

In the case of the mass concrete walls such as those of the lock, dry dock and pier to the swing bridge, it was necessary to allow for considerable settlements. In the adjacent old lock settlements of 18 to 20 cm had been recorded over a period of 35 years. (Fig. 11) In the case of the new lock unequal settlements would have prevented the operation of the lock gates. It was therefore decided to utilize as much as possible the effect on the permissible loading, of reducing the subsoil water level. (Fig. 12) Thus the unequal settlements of walls of varying heights (123 mm maximum and 21 mm minimum in two years) during construction, were as shown by Fig. 13 only reached at the present time six years after completion.

For the satisfactory operation of the lock gates it was essential that the gate rails should not be distorted, and this was ensured by only placing the rails in position just before the lock was flooded.

Road and railway tracks had to be carried over the access canal to the old harbour. Of the various systems of movable bridges, lift bridges, single and double rolling bridges, and swing bridges were considered. A swing bridge was decided on as this is least susceptible to foundation settlements. The centre pier was of massive construction (Fig. 14) in order to offset the heavy weight of the bridge by a correspondingly heavy foundation. A final settlement of 25 cm was allowed for, which in its early stages was artificially accelerated by lowering the subsoil water level. Some of the bridge bearings were arranged to be adjustable so that the exact level of the rails can be maintained in the future without great expense or loss of time.

The fourth structure (Fig. 15) on which I base my remarks, is the strengthening of the embankment in Harbour 1 in Bremen in the years 1932 -34. The ground consisted of layers of sand and gravel, with a ½ to 1 m layer of clay and peat at a lower level. As the low-water levels had fallen due to an earlier straightening of the River Weser, the timber piling had to be renewed; moreover the depth of water was no longer sufficient for the larger steamers. Instead of timber or concrete piles it was decided to employ steel piles of special type in which the low friction of sand on steel was replaced by the higher friction of sand on sand. This could, however, only be achieved if the earth within the pile section was so compressed that the pressure on the pile surfaces was higher than the friction on earth on earth. The section employed is seen in the Fig. 16 and shows at varying depths a welded flat steel point with flange. By means of this point the ground between the flanges is so strongly compressed in driving that it can no longer escape. The further advantage of this steel pile (Fig. 17) is that the position of the point and the length of the flanges can be very easily adjusted to suit existing subsoil conditions by trial drivings and loadings. As can be seen from Fig. 18 the temporary settlement of such piles even with a 120 ton load does not amount to any more than the calculated elastic compression in the steel. The loading result can therefore be described as particularly fa-

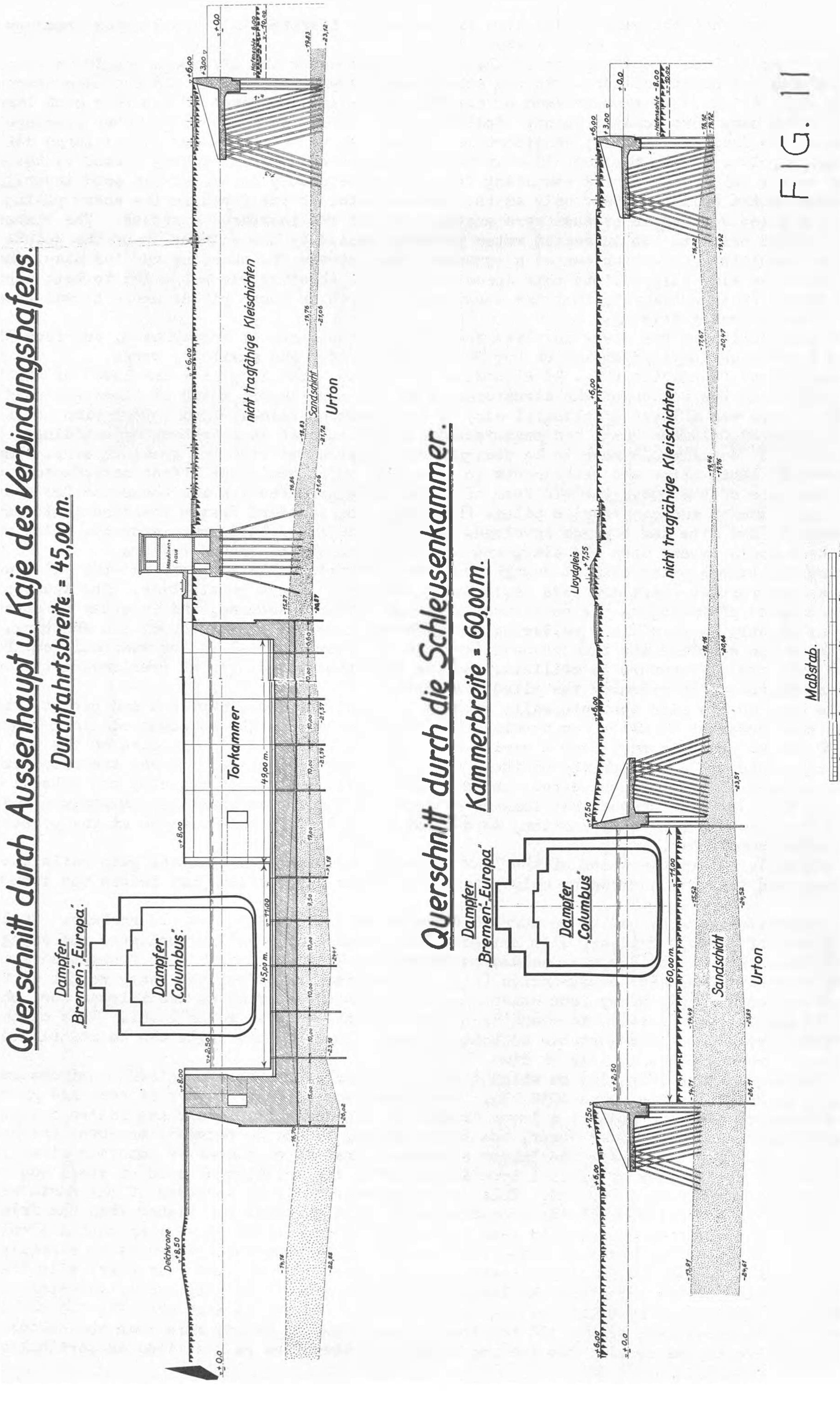

Querschnitt durch Aussenhaupt u. Kaje des Verbindungshafens.
Durchfahrtsbreite = 45,00 m.
Dampfer Bremen-Europa
Dampfer Columbus
Torkammer
Maschinenhaus
Deichkrone
nicht tragfähige Kleischichten
Sandschicht
Urton
Querschnitt durch die Schleusenkammer.
Kammerbreite = 60,00 m.
Dampfer Bremen-Europa
Dampfer Columbus
Lloydgleis
nicht tragfähige Kleischichten
Sandschicht
Urton
Maßstab.

FIG. II

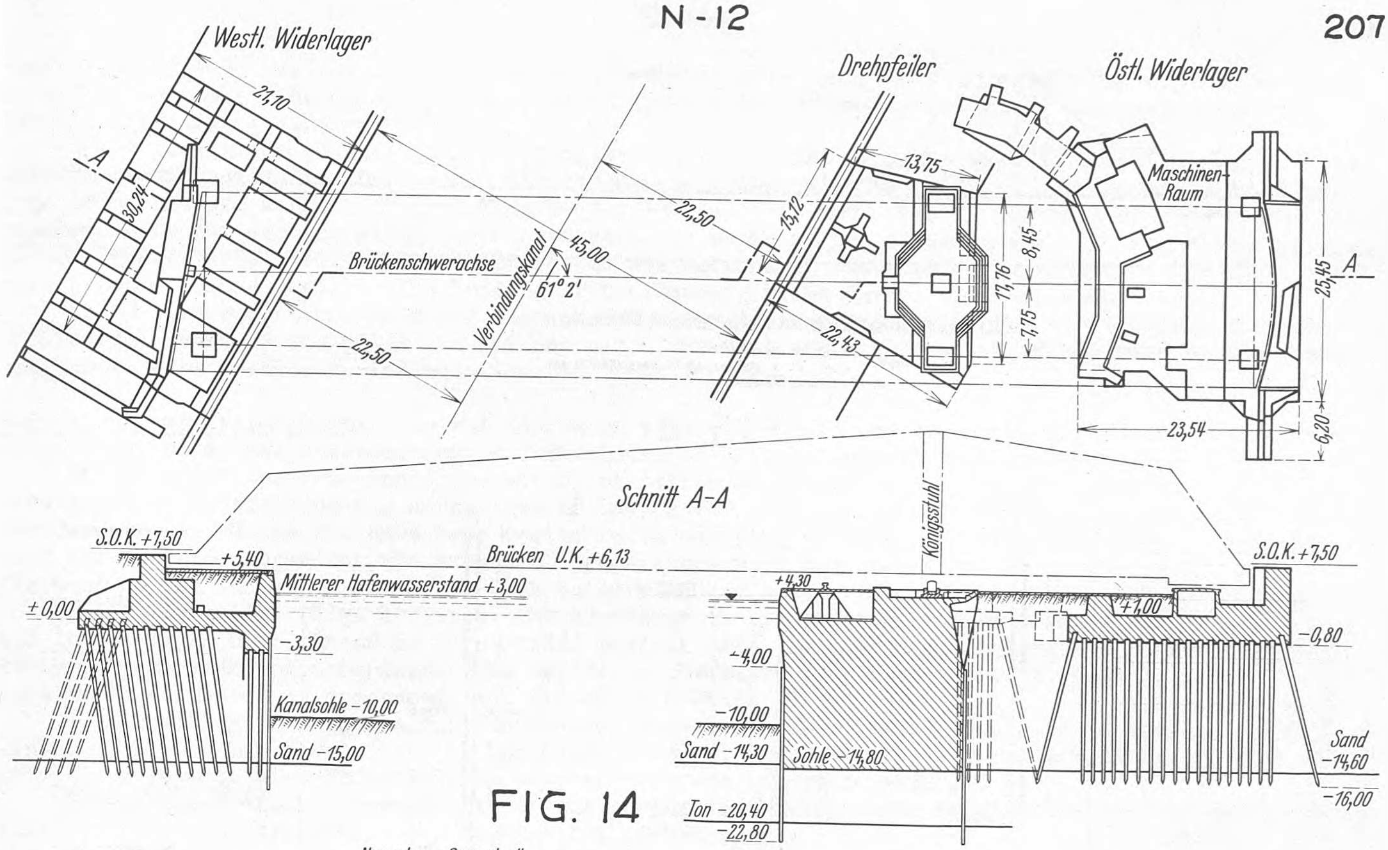


FIG. 14

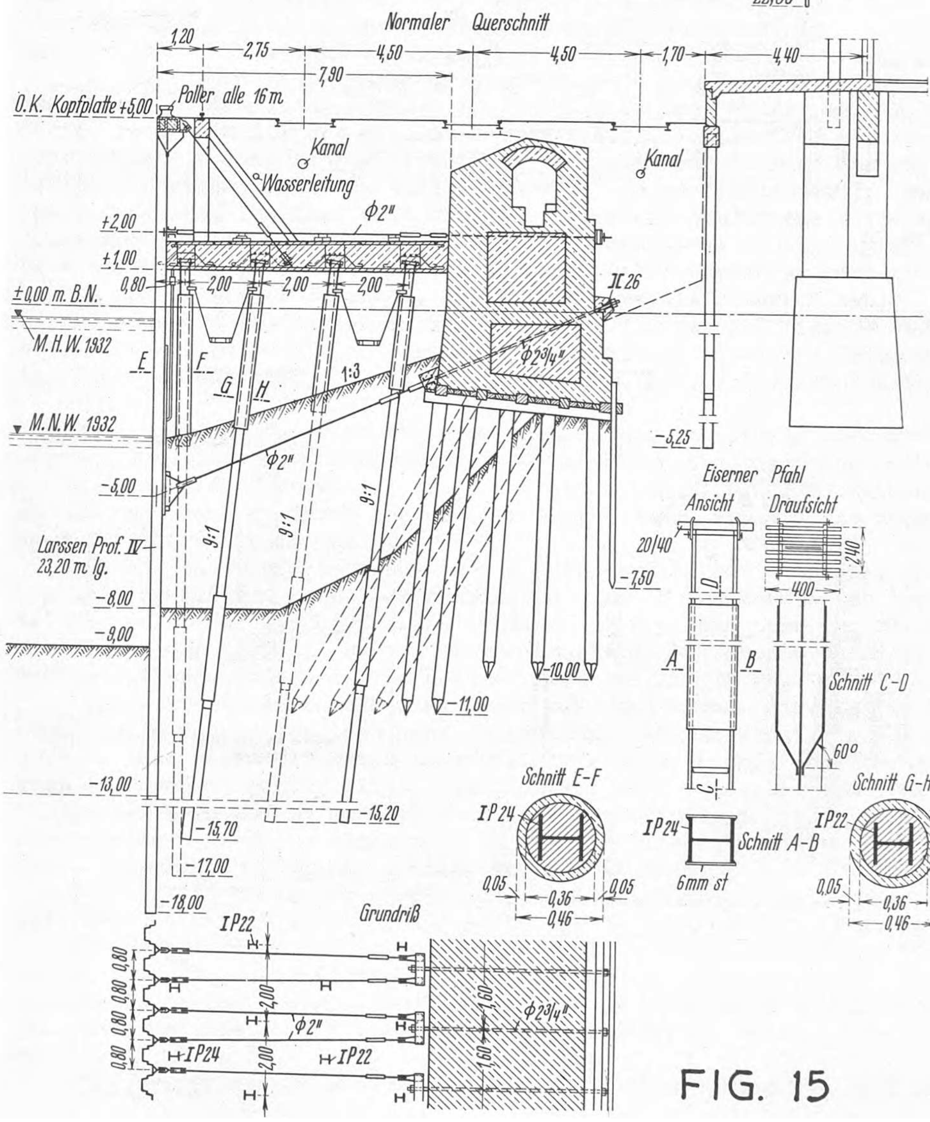


FIG. 15

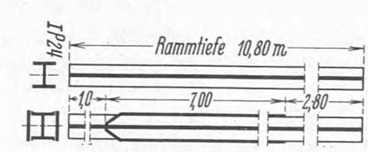


FIG. 16

FIG. 17

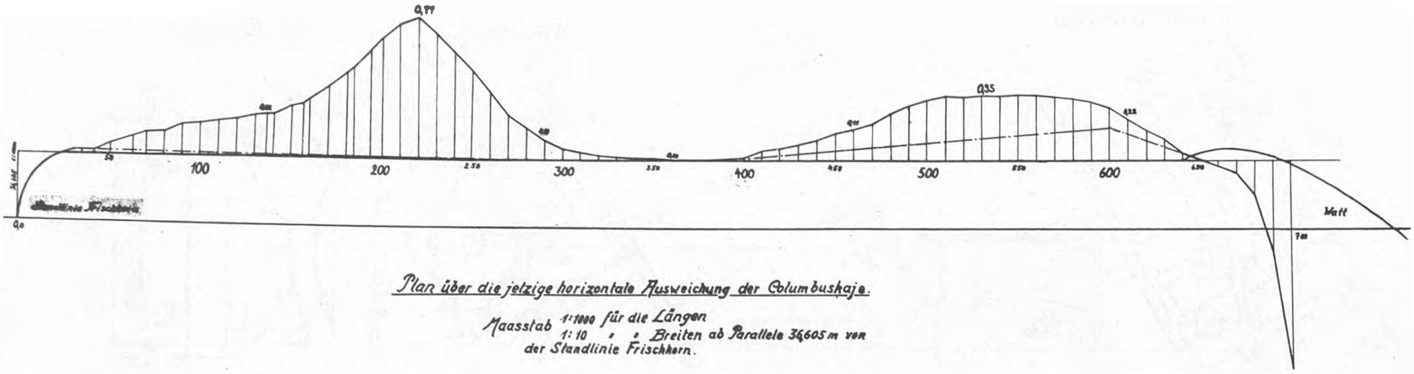


FIG. 4

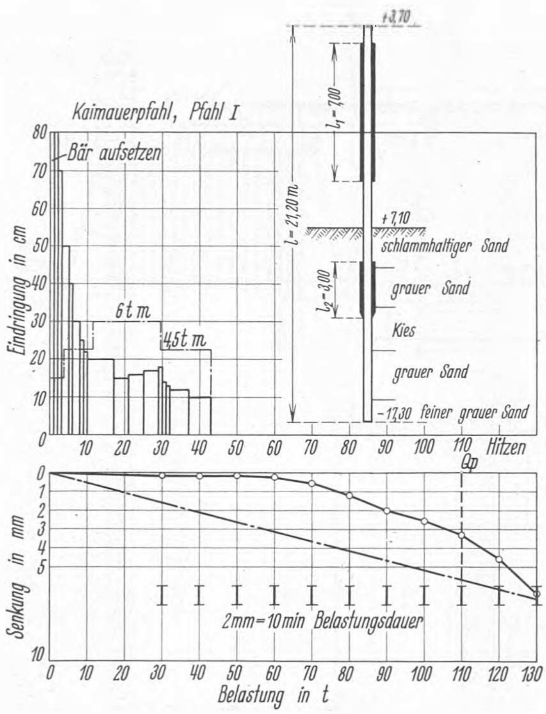


FIG. 18a

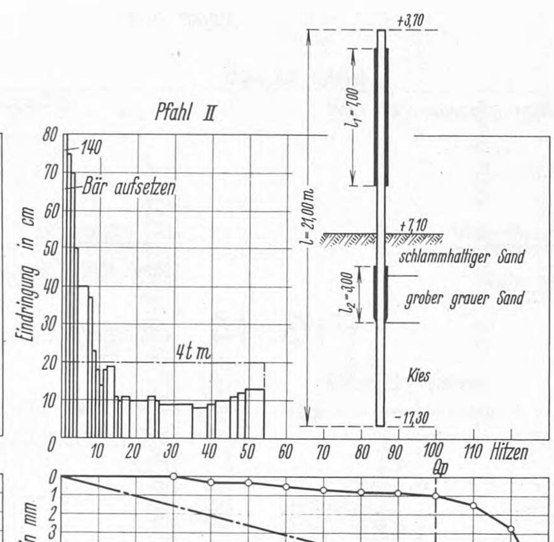


FIG. 18b

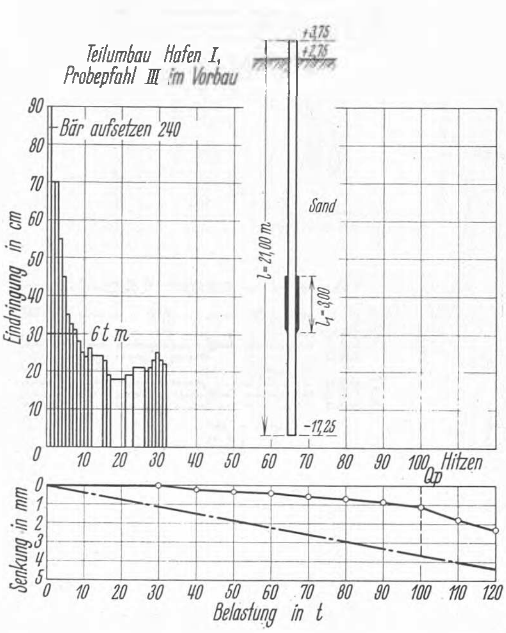


FIG. 18c

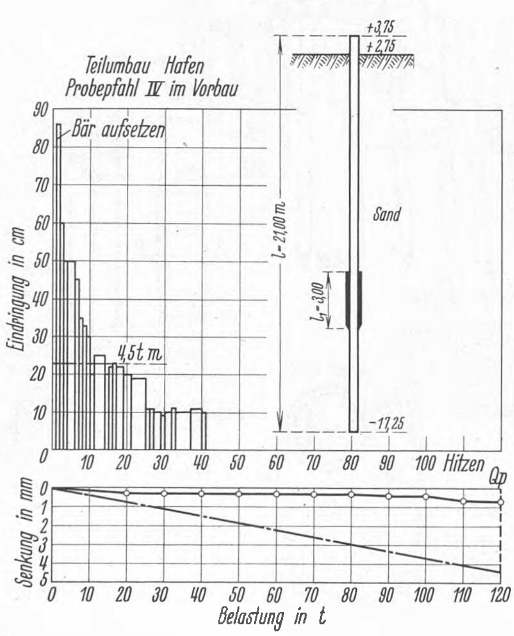


FIG. 18 d

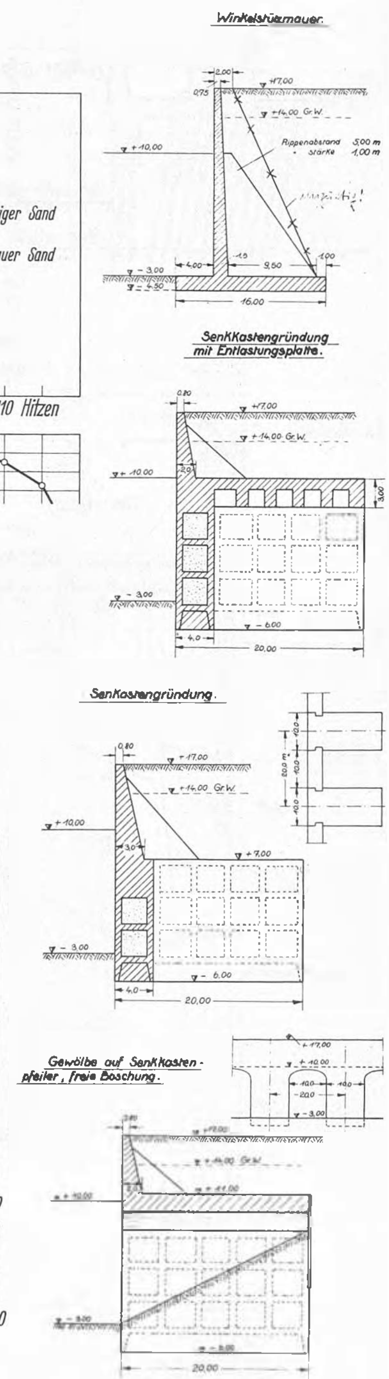


FIG. 32a

vourable. If they are not always obtained elsewhere it is in my opinion solely due to the fact that no effort has been made to ascertain by investigation the correct position of the pile point and the length of the flanges.

Apart from these pile sections, box piles and pipe piles are also used in Germany. Experiences with these in Hamburg and Stralsund clearly show that they also have a high bearing capacity. Some type of stiffening as with broad-flange beam piles has also been used in the case of pipe piles, thus strengthening the section along a definite part, as shown in the illustration. I have hitherto preferred broad-flange beams with stiffening, as the section of these piles is small and the piles therefore penetrate more easily. The loading of the Hamburg piles agrees entirely with these.

The Bremen structures have now been in operation for several years, and the loadings show that the bearing power of the piles has not decreased but that with the rusting of the sand to the steel has become even greater during the time that has elapsed.

Constructional Preparations. In the course of time experience has shown the importance of the work preparatory to actual construction, and which can be briefly described as follows:

The value of construction preparation, execution of design, and actual construction can be visualized by considering the advantages of having the same responsible Engineer in charge of all the various stages. Those who have been fortunate in carrying out the work from beginning to end agree that the decision regarding the extent of preparatory work and the selection of the design is influenced by the manner in which the work is carried out and by the requirements of the structure.

The illustration (Fig. 19) shows the advantage in starting off with a planned boring programme, and in Germany I have therefore sub-divided borings into three groups, exploratory borings, supplementary borings and main borings. The exploratory borings should at the very least be available before any design work is commenced, and a testing station should by then also have been decided on.

In the survey of the site particular care has to be taken if the ground is cohesive (Fig. 20) since experience has shown that the bench marks on which the survey is based are liable to move not only downwards but also sideways when deep excavations cannot be avoided.

In the tabulation of the results of the borings one has to bear in mind the type of construction that is under consideration. The lighter the structure and the material, and the deeper the foundation and more important the watertightness of the excavation, so the more accurate and closer together must be the borings, and the greater the area they must cover.

In determining the water pressure in static calculations one must consider the results of observations on the water level. In the case of open water one can, at least in Germany, rely upon official records which are available over a sufficiently long period.

In considering such observations one must bear in mind the extent to which water pressures will affect the structure to be built. Here it can be fundamentally stated that the smaller and shallower the foundation, the less will the water pressure conditions alter and vice versa, until the limiting condition in which the underside of the structure is located in an impermeable zone. It is necessary to abandon completely the methods by which water pressures have been treated up to now, as shown also by the researches of Terzaghi. As far as possible observed water levels should be combined with stream lines in the design in order to obtain a general view of the effect on the structure. It will not, however, be possible to avoid approximations since the determination of a system of stream lines is rather complicated, and as shown by Fig 21 subject to considerable irregularities in varying ground.

Execution of Design. When it is considered how frequently circumstances limit the extent of the designing work it is not surprising that every now and again fundamental errors are made, particularly as the standard of knowledge of ground and water is not yet uniformly high amongst engineers. When it has once been realized that both are of primary importance, more time and trouble will be insisted upon for the designing work.

On the basis of the works that I have carried out I can say that never less than 20 different preliminary designs have been considered even if I refer to one type of structure only, namely retaining walls for harbour installations. At the same time one should not as a matter of habit limit oneself to one particular type of construction, but should consider different profiles without prejudice with due regard to the cost of each per lin metre.

There are three fundamental types of foundations (Fig. 22) to be distinguished,--sheet piling, timber pile foundation, and mass foundation. If one considers the development of these one will see how the free sheet piling is converted into piled foundation through the stages of sheeting with long ties anchored to raking piles, and sheeting with relieving platform on raking piles. As the depth of the platform increases so the conditions approximate to those of a mass foundation, whilst if the platform is raised the slope will in the limiting case remain unaffected. From considerations of soil mechanics sheet piling has the advantage that the vertical loading on the ground is not increased. The sheeting is therefore hardly subject to settlements quite apart from the fact that settlements will have no effect on it if the anchorage is sufficiently flexible.

The application is limited by the quality of the ground. A low angle of repose increases the length of the ties considerably so that the cost of construction is very high. The steel industry admittedly is now able to provide very strong sections of piling so that the construction of a sheet pile wall becomes more a question of the necessary anchorage than of a sufficiently high section modulus in the wall.

The tying back of sheet piling (Fig. 23) has during the last few years also been considered in

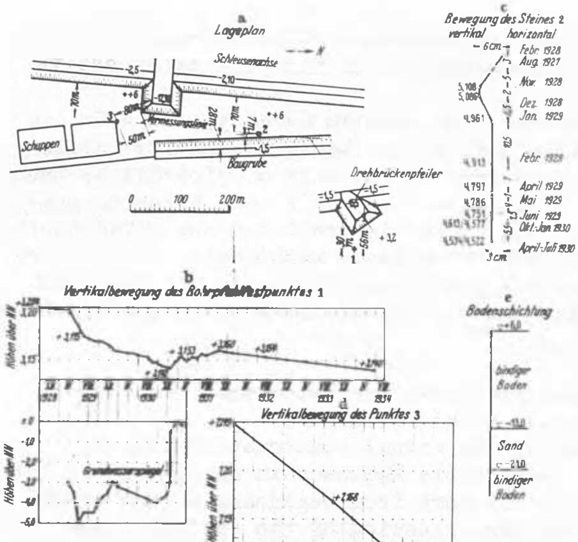


FIG. 20

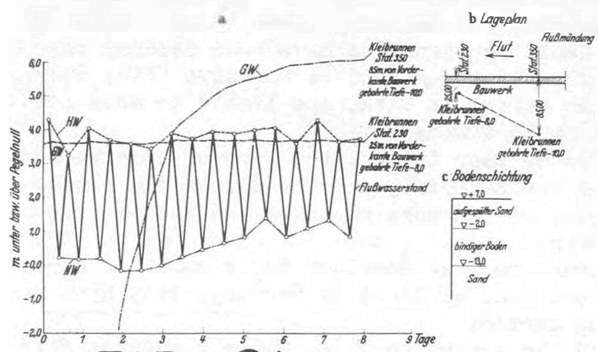


FIG. 21

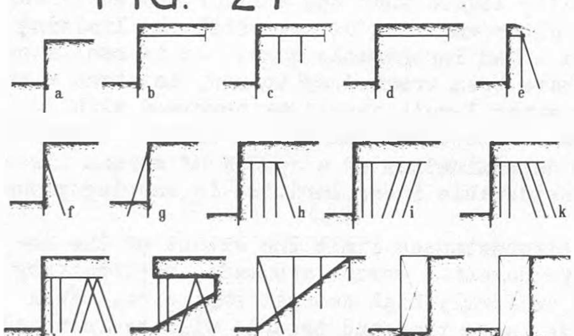


FIG. 22

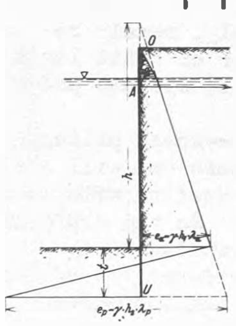


FIG. 23

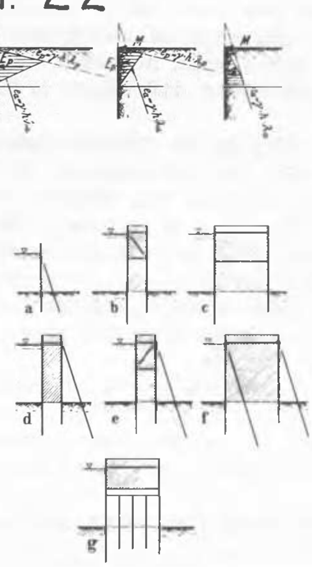


FIG. 24 (right)

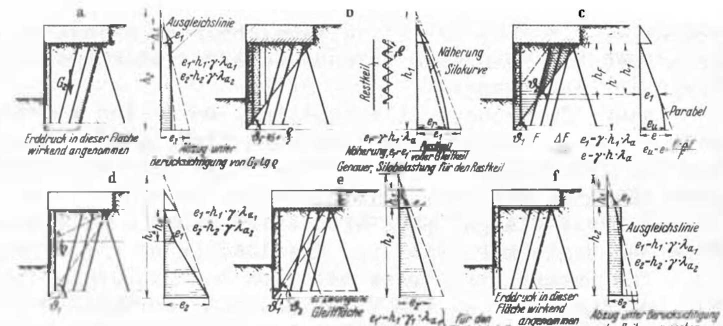


FIG. 25

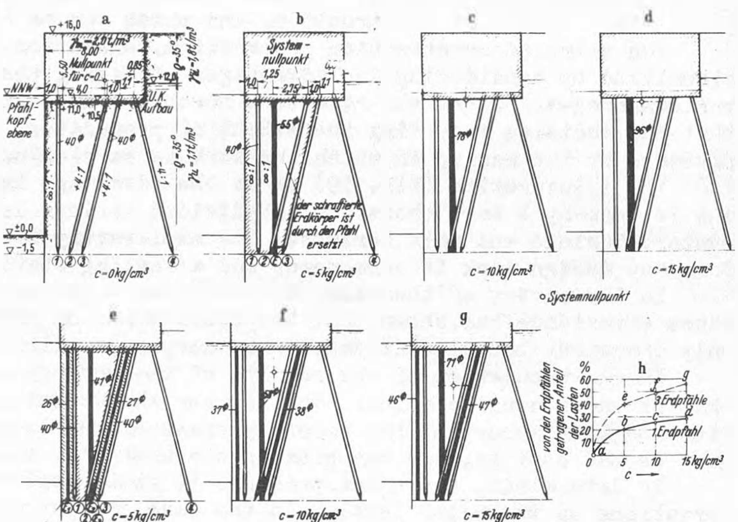


FIG. 26

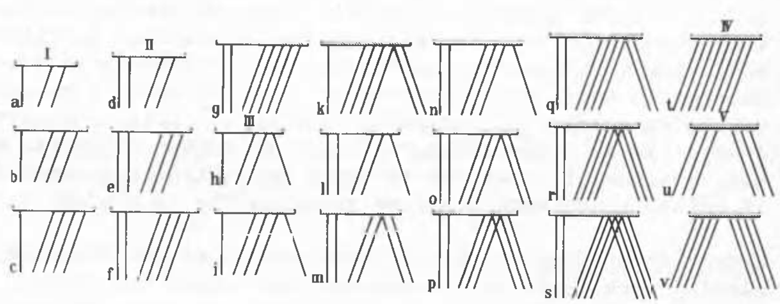


FIG. 27

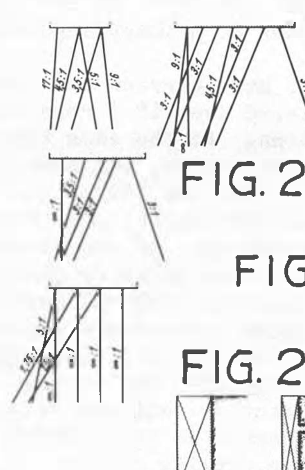
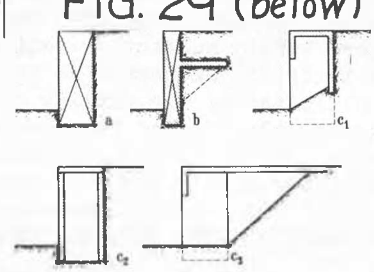
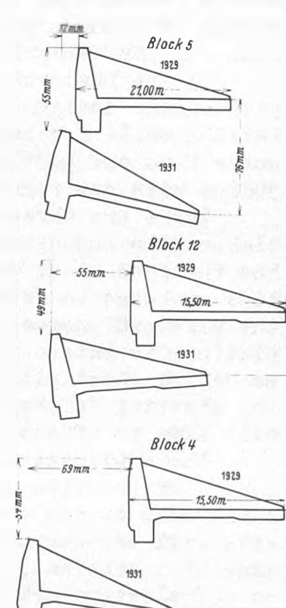


FIG. 28 (left)

FIG. 33 (right)

FIG. 29 (below)





Germany from another point of view, as in a number of cases the round tie-rods have failed. Researches have shown this to be partly due to the settlement of the ground under the ties causing an excessive stress in the steel. Further, careful calculations of the sheeting have shown that with an exact consideration of the support at the top end of the sheeting, passive earth pressure can occur. This causes a certain amount of fixity in the wall which, although negligible so far as the piling is concerned, does cause an appreciable increase in the reaction at the support, this reaction being taken up by the tie-rod.

If the length of the ties exceeds 15 - 20 m one should therefore try and avoid using them by anchoring the sheet piling to a relieving platform, the width of which would be very much smaller than the length of the necessary ties.

Another method of shortening the length of the ties (Fig. 24) is to make the anchorage act as a cantilever fixed in the ground below the slope of repose. This causes an increase in the dimensions of the anchorage according to the depth until one eventually reaches a condition in which the structure consists of two parallel walls of sheeting driven to the same depth as applied up to now for double skin cofferdams. The hope that the earth wedge between two lines of sheeting would exert considerably less pressure on the front wall then the theoretical amount has not been realized. Thus a very much greater quantity of material becomes necessary for this type of structure.

Another consideration leads to the anchoring of the piling not by tying it back but by driving anchor walls at right angles to the sheeting so that they obtain a grip on the ground at the back. It is then assumed that owing to the friction of the ground in the corrugations of the anchor wall, the weight of the material in these corrugations can be taken to exert a reverse turning moment on the wall. An appreciable advantage of this type of construction is that the anchorages are not affected by settlements, and that the danger of failure in tension is eliminated.

When considering structures consisting of two lines of sheet piling it is essential to distinguish whether the ground between the walls is to be employed for taking up the earth and water pressures or whether the ground and the steel are to act in combination with one another. This still presents a problem for research on soil mechanics as it has not yet been definitely determined in what manner the slopes of rupture and of repose form inside a double-skin dam, also what pressure the soil particles exert and what movements of the wall are to be laid down in the Coulomb earth pressure theory. There is much uncertainty in calculations for double-skin dams, and for the time being assumptions have to be made which err on the side of safety and which are therefore uneconomical.

Theories based on the arching effect of the ground prove to be unsuitable when considered more closely. A number of superficial investigations which I have carried out on various assumptions showed considerable variation and finally led to the conclusion that here the limits of the application of classic earth pressure theories have been reached.

Piled foundations lend themselves on the other hand to more exact calculation and have therefore been used much more frequently than structures based on a double line of piling. I will not, however, deal in more detail with methods of calculation for piled foundations as they have already been fully developed and do not require to be dealt with more closely.

A further question which has not yet been settled in the design of high piled foundations is the effect of the carrying capacity of the ground itself under the pile cappings. Up to the present this has been neglected altogether and thus a simple basis obtained for the design. This, however, is not entirely correct. Admittedly it is on the safe side as the loading of individual piles is greater if the carrying capacity of the ground is neglected. On the other hand the loading on the wall of sheet piling at the front would be less favourable as the effect is similar to that of a superload. I have, however, attempted to determine the distribution of the load between the ground and the piles in an example, and have come to the conclusions shown by Fig. 25.

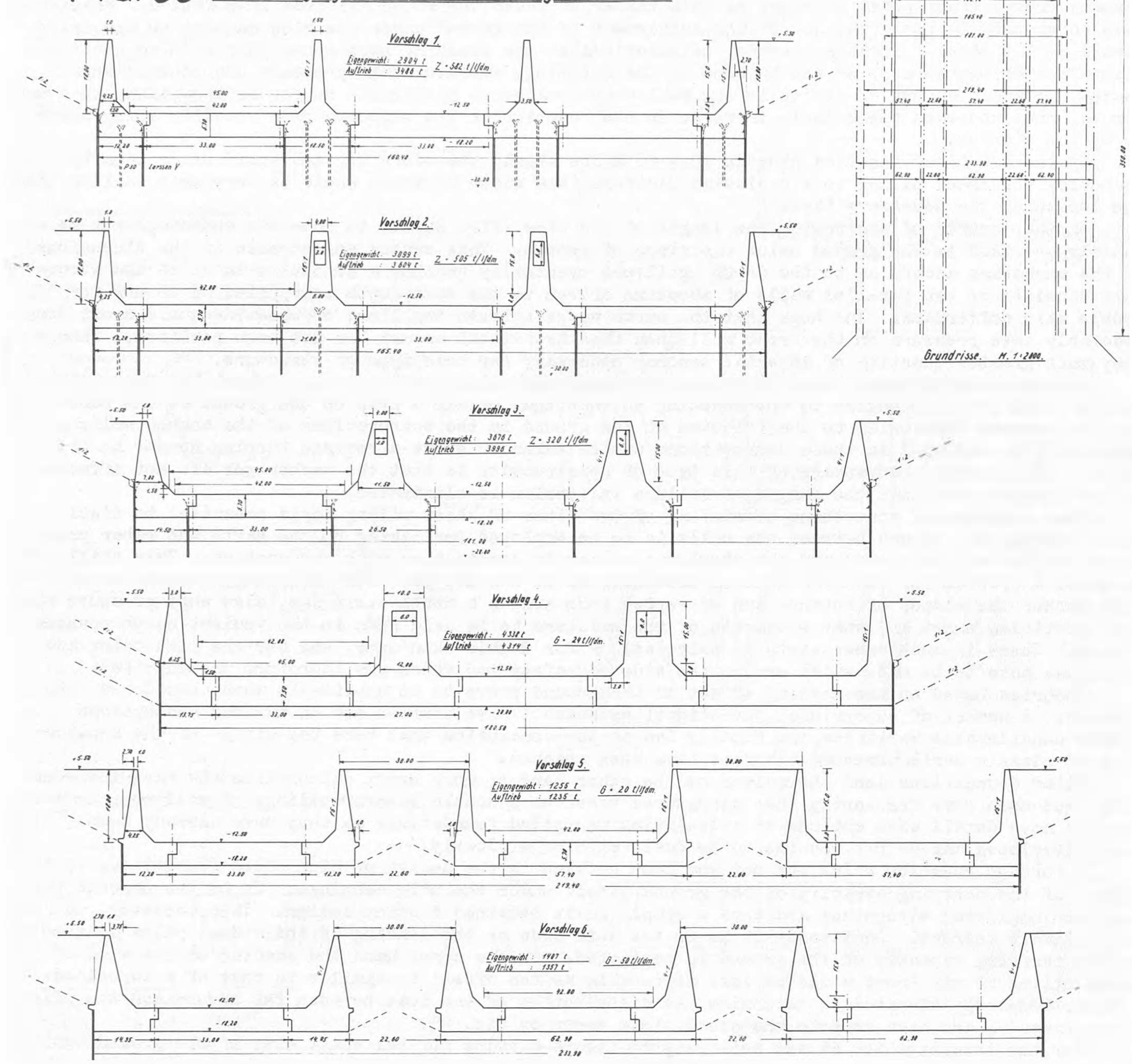
For the investigation it was necessary to assume a value for the elasticity of the ground and for trial purposes I have taken a loading coefficient C of 5, 10 and 15 kg per cu cm. (Fig. 26) The investigation was chosen in such a manner that one could freely apply the calculation method of Nökkentved. The ground itself was divided up into individual piles which were treated in just the same way as if they consisted of another material. The differences in the results regarding the choice of the coefficient C, and the manner in which the ground was divided up into one or more piles are appreciable.

It is, therefore, to be concluded that to obtain accuracy the subdivision must be as small as possible, and further that a practical application of the investigation must be delayed until researches in soil mechanics can give us with certainty the values of C to be used in the calculation. It is of course assumed that the elasticity of the piles (Fig. 27) is greater than that of the ground so that to the elasticity of the material forming the piles must be added the elastic settlement of the pile cappings.

In the design of pile foundations I have successfully applied the principle of making the distribution of the piles as free as possible. (Fig. 28) It is only necessary to examine the pile foundation systems previously carried out in order to realize that a concentration of pile points in a small area, the driving of piles at different batters very close to one another, and also the driving of piles at very close centres must lead to an unfavourable transmission of the loading to the ground.

In the case of mass walls I differentiate between piers subject to vertical loading, and retaining walls. The latter are of much greater interest, and Fig. 29 shows their diagrammatic arrangement. In these structures one would make the attempt to reduce the horizontal earth pressure by the use of relieving platforms, and to reduce the vertical pressure by spreading the structure over a

FIG. 30 *Vorentwürfe für eine Trockendockanlage. Maßstab: 1:500.*



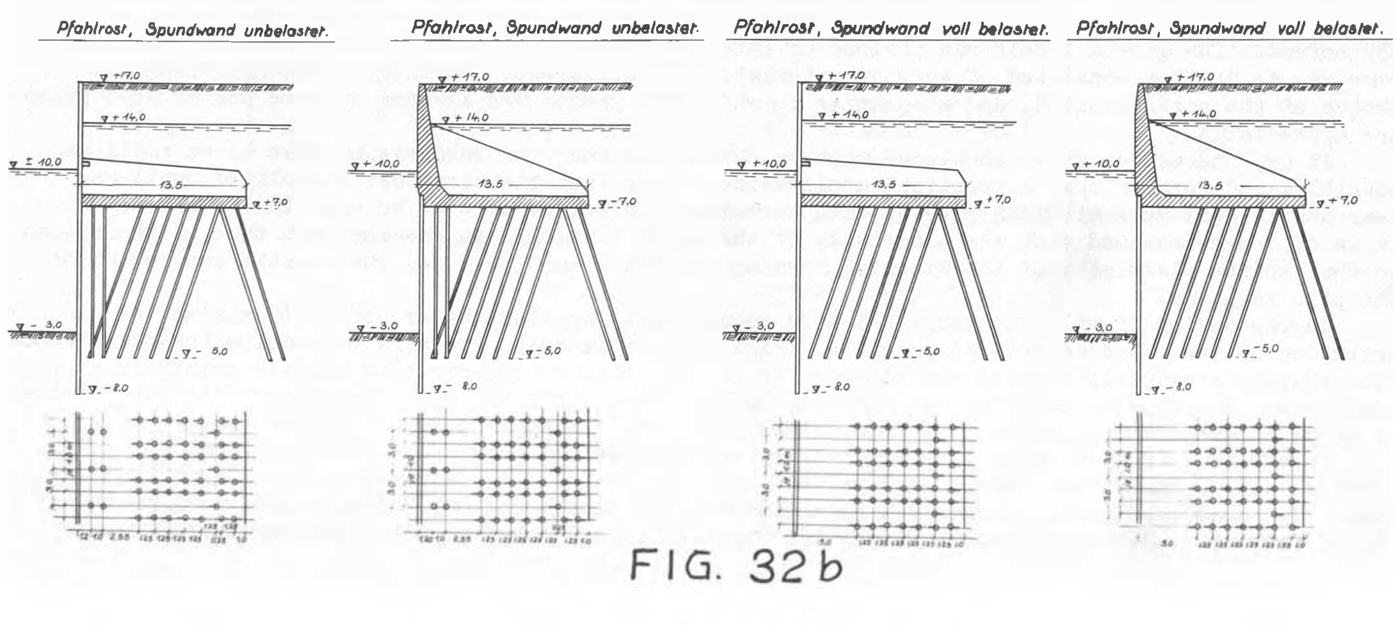


FIG. 32b

greater area, when settlements can be anticipated. From both these requirements there follow various possibilities for the division of the structure into individual piers, the addition of relieving platforms and utilization of reinforced concrete as a weight-saving material.

Finally there are the trough-shaped structures which are used for dry docks and underground railways, and which are shown by Fig. 30 in their various forms. Here each individual case will have to be examined for the constructional and economical limit between a mass structure which is in itself safe against upward water pressure, and a reinforced concrete structure which has to be protected against such water pressure by sheet piling or by tension piles.

Having briefly dealt with the various types of structures which have to be considered in the preparation of a design I would now like to illustrate with the aid of an example the various designs considered in relation to a particular problem.

The structure concerned is a retaining wall 20 m high founded on ground that is mainly sandy but containing clay strata. Operating conditions and the low permeability of the ground necessitated an allowance in the design of 4 metres for the water surpressure. The attempt (Fig. 31a) to design the structure in sheet piling was unsuccessful owing to the expense of the 46 m long tie-rods that would be necessary. The possibility of forming a double wall of piling is shown by Figs. 31 b & c in which the stiffening is provided partly by sloping or horizontal steel sections, and partly by a capping of reinforced concrete. In fitting the steel sections the loading was to be transmitted to the ground so that the steel would be stressed in tension. In this manner one could employ a standard system and thus eliminate the difficulty of more detailed consideration of the formation, planes of slip, and of fixation, in the fill. The design obtained is, notwithstanding the severe assumptions, not so unfavourable as to be rejected immediately. Construction of crosswalls of piling was, however, found to be less economical. It was therefore attempted to eliminate the front line of piling and to slope the ground up. The cost is here still rather high and it is found to be more economical to drive individual steel piles into the ground, thus converting the structure into a type of pile foundation. (Fig. 32a) At this stage I abandoned the idea of carrying the piling up to the top of the wall and decided to replace the anchorage by raking piles. The structures shown in Fig. 32b were thus obtained. In one case the sheet piling was not subjected to vertical loads at all. In the other case it was subjected to them by a pre-determined amount. For reasons of efficiency and economy the question was considered whether the upper part of the sheet piling should be replaced by reinforced concrete, and two corresponding cross-sections were thus obtained, the cost of which is a trifle lower than with steel piling for the full length and height.

Finally, design possibilities in mass construction were considered, once in reinforced concrete with economical results; there was also considered construction with the aid of wells or compressed air, with and without relieving platform, and with arch or free slope. The latter method is somewhat uneconomical as the space between piers is small relative to the width of the piers, as ground pressures otherwise become too great. In other localities this method has been more successful. It should be remembered that owing to the high water surpressure a reduction in earth pressure is not as important as elsewhere.

Construction. The effect of settlements on various structures is shown by observations made in Bremerhaven. (Fig. 33) The manner in which movements take place in the blocks of a retaining wall on piles I have had reproduced on an exaggerated scale. It will be seen from this how irregular the movements of individual blocks are in a forward or backward direction in the same retaining wall. Other illustrations (Fig. 34 a b c d) show the influence of individual constructional processes on the movements of the structure. Diagrams also show the measured movements of piled foundations built on sand strata of various thicknesses. The influence of lowering ground water level is clearly seen and it will be noted that thereby the pre-stressing of the ground was quite appreciable. Later on the settlements were in some cases actually reduced although the loading caused by the structure remained constant.

The next illustration (Fig. 35) shows the settlement of a building on a raft foundation. There is little to say about this and after seven years the maximum settlement has not yet been reached. The extent of the settlement depends less on the loading than on the nature of the ground.

Movements of a mass concrete swing bridge pier and of two abutments on piles are seen from Fig. 36 which shows that the difference between the heavy pier and the two abutments is not really particularly great. Here also the characteristic rise of the structure was observed after the lowering of the subsoil water was discontinued. The same impression is obtained from the following view which shows settlement of a deep mass foundation, and which differs from the previous view only by the amount of settlement.

Management. With regard to the management of structural work I again emphasize the importance of continuous control. Without this we would not today be in possession of our present knowledge. And how much further would we be in our knowledge of soil mechanics if similar measures were adopted by all engineers in the world.

Progress in Construction and the Future Development of Constructional Design. If one surveys once more the developments that have taken place not merely in the 8 years during which the structures described above were built, but in the last 20 years, one will observe the attention paid to foundation problems; as a result we are able to erect heavy structures with more certainty on weak ground.

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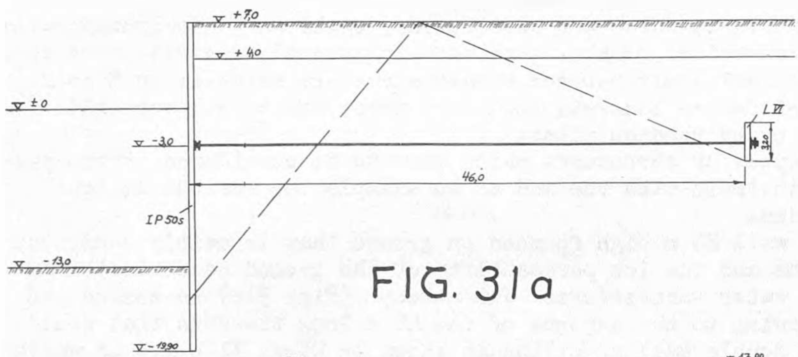
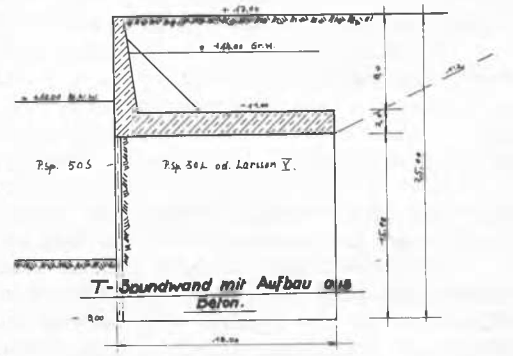


FIG. 31a



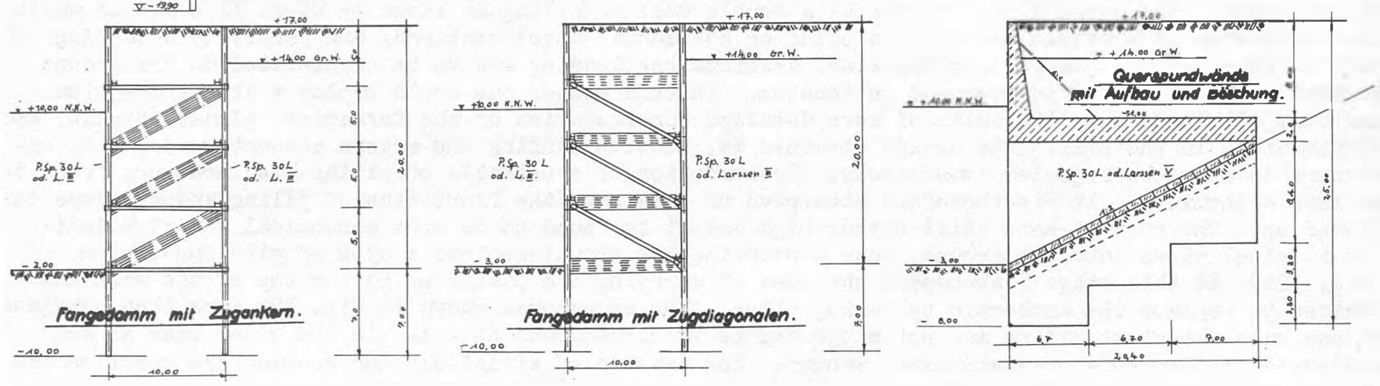


FIG. 31c

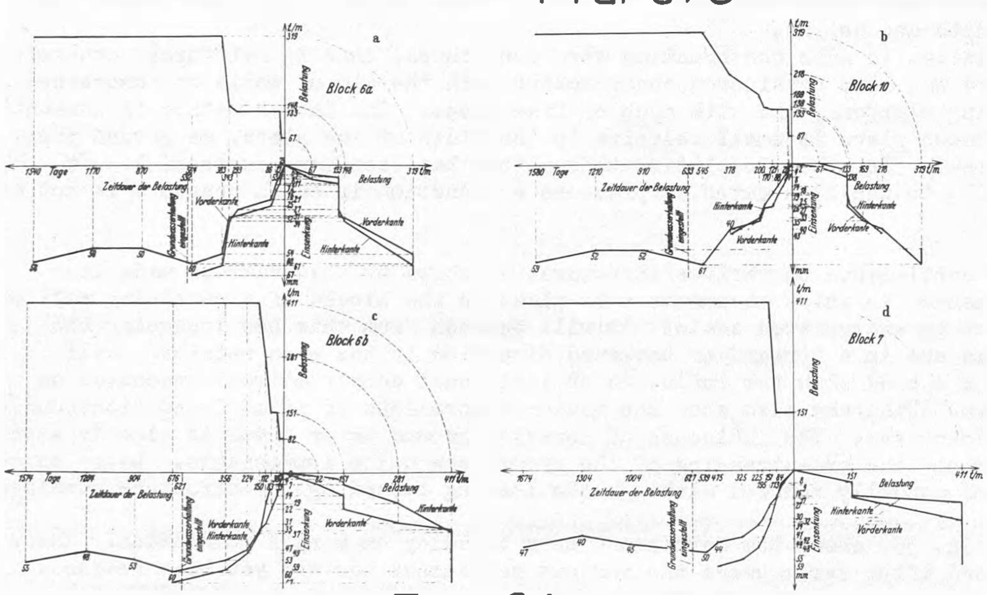


FIG. 34

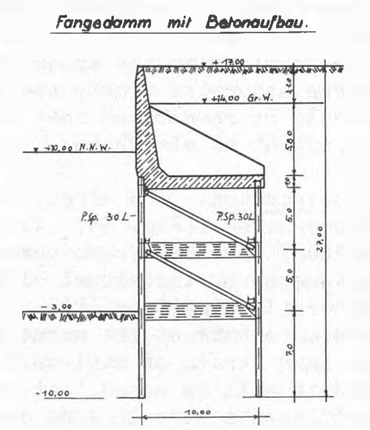


FIG. 31b

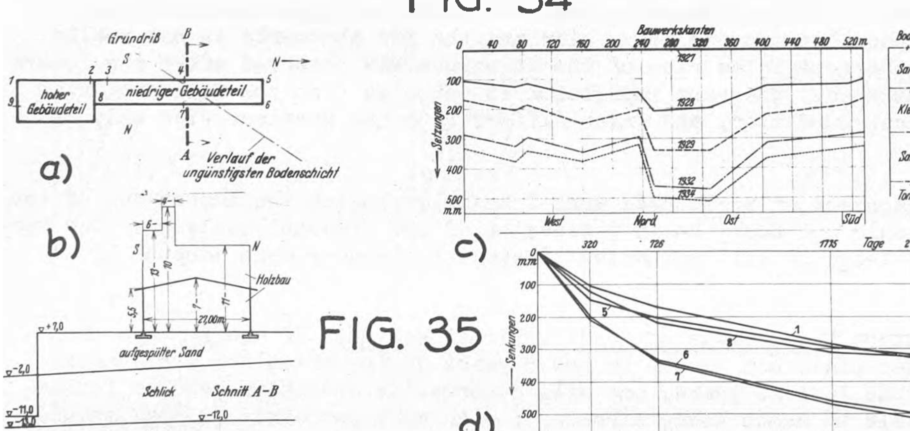


FIG. 35

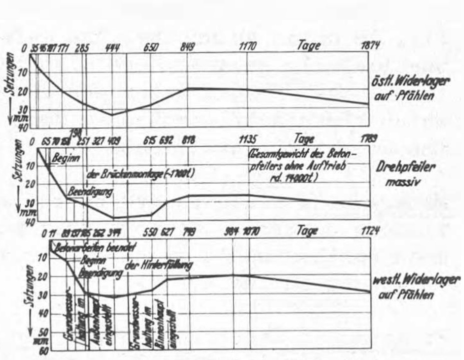


FIG. 36

One must not underestimate the value of such developments, which for instance enable us to calculate the carrying capacity of piles not from empirical formulae but to base them on trial loadings with a definite value for t. Similarly it must be recognized that it is now customary to calculate earth pressures not from tabulated information, but from the results of tests. Unfortunately there is a gap between the requirements of soil mechanics and of practice in so far that the complicated calculations for, say, clay pressures are not easily accessible to more practical engineers. Moreover, investigations with the aid of soil mechanics have become so specialized that they make it impossible for the constructional engineer to arrive at his own decisions independently of the laboratories. That is naturally a condition which has considerable drawbacks.

Future developments can perhaps be characterized by expressing the hope that the uncertainty exposed by researches in soil mechanics will be eliminated as the results of tests accumulate. It may then soon be possible to reduce these to general formulae which would be accessible to all and would appear in such simple form as to be understood even without special knowledge of the subject. It will then be possible to make much more easily a critical comparison of various foundation methods and thus to solve their various problems more satisfactorily and economically.

No. N-13

THE FOUNDATION OF THE BUILDING "LA BALOISE" IN LUGANO, SWITZERLAND INVOLVING MODERN METHODS IN DEEP FOUNDATION TECHNIQUE

G. D. Rodio, Consulting Engineer, Milan, Italy

Introduction. The Swiss Insurance Company, "La Baloise", planned in the year 1933 the construction of a building for their Branch in Lugano, and charged a Swiss Building Contractor with the execution of the work. This new building, which was erected during the years 1934 and 1935, has nine stories, two of which lie beneath the street level. The area built on measures approximately 1200 sq m, and the position of the building is shown in Fig. 1 and 2.

At a distance of approximately 100 m from the building is situated the Lake of Lugano, whose level varies between 272.0 and 269.5 m above sea level, while the ground level is approximately 273.5 m. The deepest excavation of the two-storied basement had to be made to a depth of 264.2 m above sea level so that the construction of the foundation trench could resist a water pressure of 7.8 to 5.3 m.

After the completion of relatively few and insufficiently tested borings, the General Contractor drove down, round the foundation trench, a 10 to 12 m deep Larssen sheet piling; it was then intended, while pumping out the water, to excavate the foundation trench in order to drive down from its dried floor reinforced concrete piles on which would rest the weight of the building.

This program could, however, only partially be carried out according to the manner planned, as, in the course of the work, great difficulties were encountered, and a continuation of the foundation work by normal methods could not be considered. The assistance, therefore, of the Firm S. A. Ing. G. Rodio & Co., Specialists for Foundation Work, had to be sought.

Thanks to the energetic initiative of the architect, Paul Vischer, and the consulting engineer, Dr. hon. c. E. Gruner Basel, the difficulties could be overcome involving modern methods of subsoil treatment. Mr. Vischer and Mr. Gruner insisted upon carrying out the original program not restricting the construction as used under all the other buildings in Lugano to one or no cellar at all.

Description of the Construction Process Previous to the Participation of the Specialized Firm. (See Fig. 6) From the results of the borings made with every care by the Specialized Firm, it appeared that the soil consisted on an average of the following layers: Approximately between the levels

273.5 - 263.0 .. Gravel and Sand
263.0 - 261.2 .. Gravel, Sand and Silt
261.2 - 249.0 and over - Fine Sand and Silt.

This last layer could be considered in practice as impermeable for foundation purposes.

As a result of the borings, which were carried out before construction was started, the layer between 261.2 and 263.0 m was considered sufficiently impermeable to fix therein the lower end of the sheet piling. As was later shown, this was a fatal mistake.

Even at the beginning of the insertion of the sheet piling and the excavation of the foundation trench to 2 m, the neighbouring buildings showed signs of cracking, so that steps had to be taken to underpin them. After $1\frac{1}{2}$ months' work, the driving down of the chief sheet piling and a secondary sheeting was practically completed, and the excavation had proceeded so far that water had to be drained away while the work continued. At the same time, two reinforced concrete piles were driven in for testing purposes.

The vibration caused by the insertion of the sheeting and piles, as also the disturbance of the underground soil by the forceful flow of water under the sheet piling during the drainage process, produced in the neighbouring buildings a settlement of approximately 7 cms and a slope of the wall towards the foundation trench of approximately 6 cms. This resulted in very noticeable cracks in

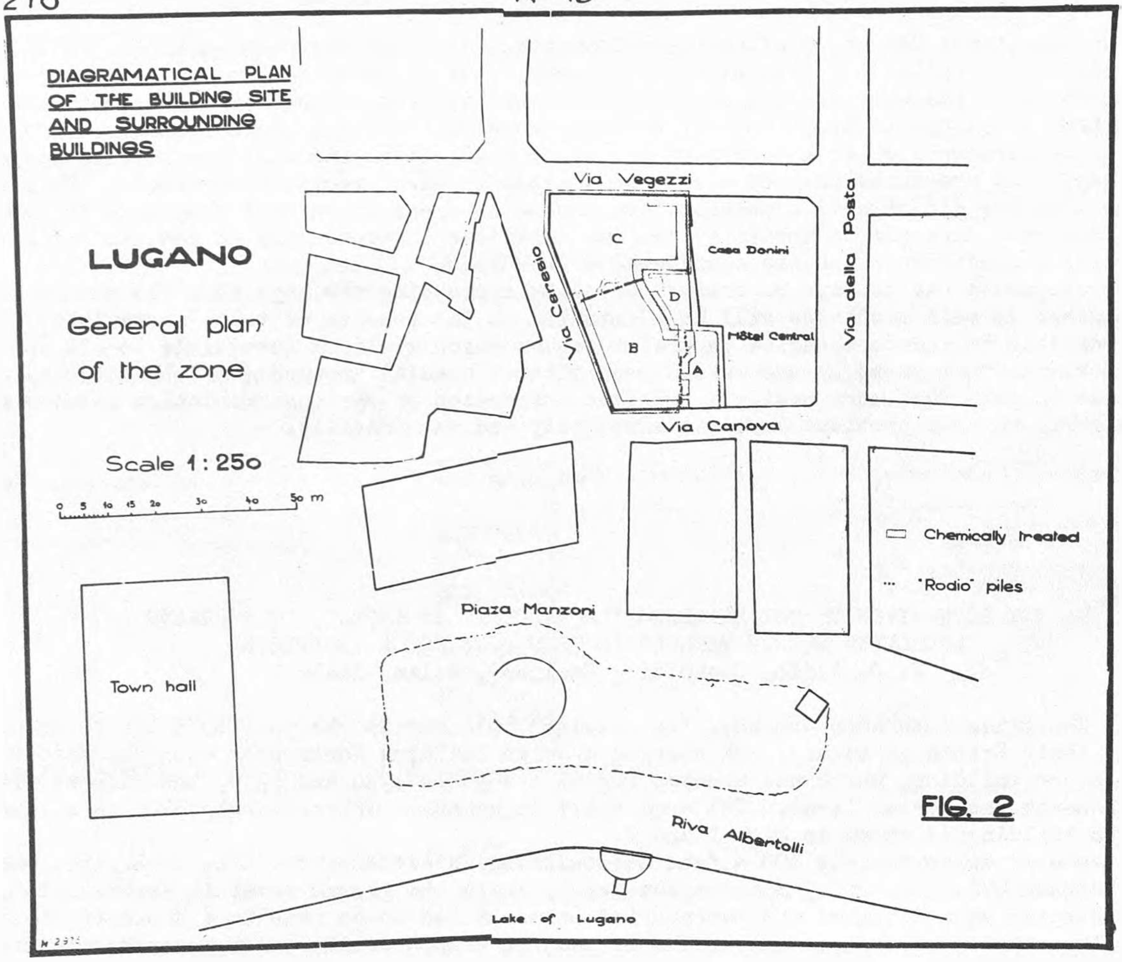


FIG. 2

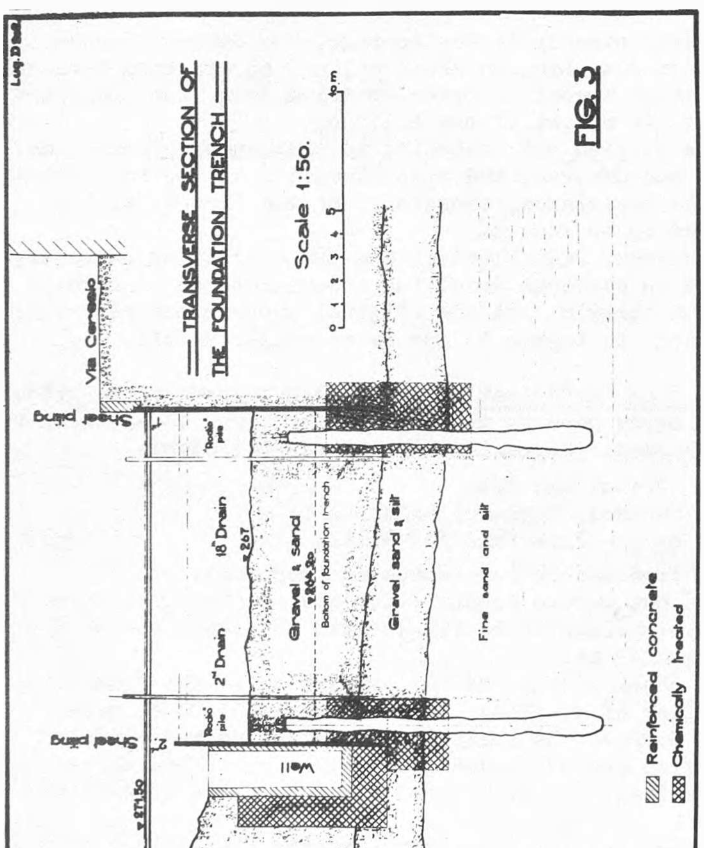


FIG. 3

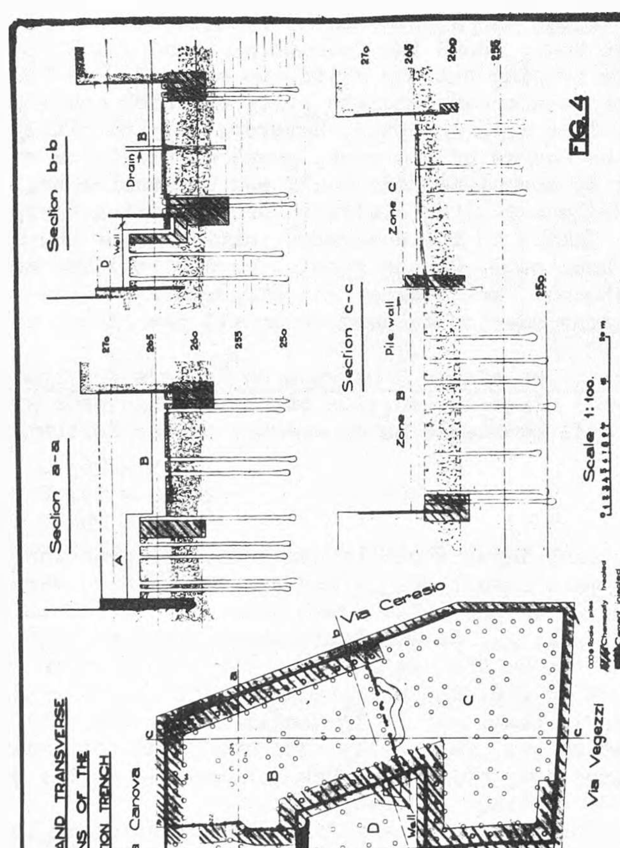


FIG. 4

the wall-work; thus, for instance, a crack of about 8 cms width could be observed!

After further extensive safety measures and underpinning work had been carried out in relation to the endangered buildings, the driving in of the sheet piling was completed and efforts were made to proceed with the excavation work.

Five months after the start of the work serious changes appeared within the foundation trench and in the neighbouring buildings so that a continuation of the work by normal means was no longer possible.

The sheet piling inclined in certain places approximately 9 to 10 cms towards the foundation trench, while the ground lying behind settled at 12 to 15 cms.

The upper ends of the piling were bound by strong, 40 cms, Peiner girders and the foundation trench was stiffened by an iron and wooden framework lying one above the other.

A last effort in which the excavation work proceeded with the help of a Mammut pump, had immediately to be interrupted as in one place the slab-floor of the foundation trench broke up in the form of a funnel and a small issuance of gas took place, during which water, organic particles and fine sand were thrown up.

After this last failure the Specialized Firm was asked to co-operate.

Proposal of the Specialized Firm for the Consolidation of the Foundation Trench and its Immediate Surroundings. In agreement with all concerned the following program of work for the consolidation of the foundation trench was drawn up:

(1) The use of rammed in piles was to be finally abolished. In their place, for the support of the vertical loads, Rodio Piles (those bored into and moulded in the ground itself) were applied, in order to avoid any shaking of the less stable ground. At the same time there was the possibility of obtaining while boring the piles undisturbed samples of soil and thus further safety measures could be conducted.

(2) The foundation trench was to be divided into three zones, A, B, and C (See Fig. 4). These divisions, which would be made by sheeting of closely lying Rodio piles, would have the object of enabling partial excavation and thus avoid possible local difficulties spreading over the entire building area.

(3) Under the sheet piling, near the buildings immediately touching the foundation trench, injections of cement were to be made in order to prevent ground movements and to close the worst cracks.

(4) The Zone A was to be left at a depth of 267.0 so that here only one floor in the basement would be built.

(5) For the lowering of the excavation of the sections B and C from the existing depth of approximately 268.0 to the previously mentioned depth of 264.2, two proposals were put forward:

Proposal A. By means of the introduction of a water-tight, chemically consolidated slab between the depths of 263.2 and 264.2, which should be perforated by gravel and pipe drainage through which water could flow away without the fine particles being carried with it. The drainage should be so arranged that the upward pressure of the underground water on the consolidated slab is not larger than the weight of the slab itself. These conditions are shown in Fig. 7.

Proposal B. By the extension of the iron sheet piling by means of the injection of chemicals in order completely to cut off the inflow of water into the foundation trench and thus enable continuation of the excavation work without the necessity of draining away water (See Fig. 3, 4 and 8).

At this point it must be mentioned that, after intensive testing in the Firm's Laboratory at Milan and on the building site itself, the first solution (Proposal A) proved to be too difficult in practice so the stoppage process outlined in the second proposal was carried out.

While the work of inserting the piles on the building site was immediately begun (See Fig. 9), experiments for the clarifying of the theoretical and practical problems, which arose in connection with the special circumstances at Lugano, were performed in the Firm's Laboratory in Milan.

Examination in the Laboratory of the Theoretical and Practical Bases for the Program of Work.

1. Soil Mechanics Analysis. The routine tests were then made on the soil samples. For each sample, the following were determined:
   1. Test for the coefficient of permeability - K.
   2. Consolidation test.
   3. Shearing test.
   4. Atterberg limits.
   5. Grain sizes.
   6. The specific gravity of grains.

In Fig. 5 three particularly characteristic results are indicated.

The above tests gave exact data for determining the dimension of the piles and for the calculation and designing of the zone of consolidation.

From comparison of the several borings it could be stated that the upper surface of the practically impermeable layer (which had an average value for $K^{10} = 0.06 - 0.002 \times 10^{-4}$ cm/sec lay approximately 261.2 m above sea level. Above this lay a sand layer of varying thickness which was actually more permeable ($K^{10} = 2.0 - 32 \times 10^{-4}$ cm/sec)and was covered over by a gravel and coarse sand layer whose value for K^{10} ranged from $16 - 200 \times 10^{-4}$ cm/sec, and in some places even up to 1000×10^{-4} cm/sec.

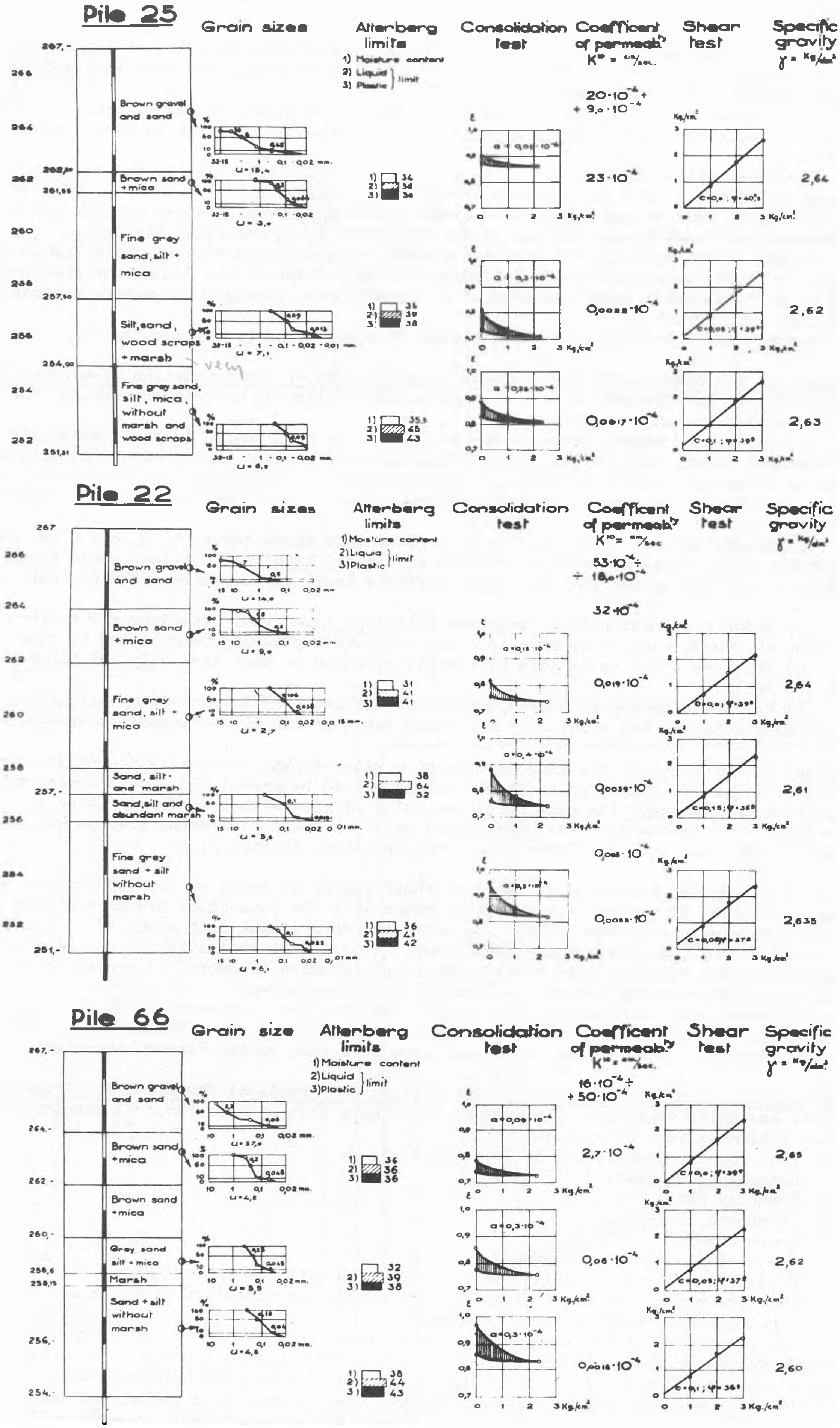


FIG. 5

SOIL MECHANICS ANALYSIS

The sheet piling reached everywhere to the layer where $K^{10} = 2.0 - 32 \times 10^{-4}$ cm/sec, and at some points even went into the practically impermeable zone.

2. Analysis of the Static Conditions of the work of consolidation in question by means of Calculation and Experimental models.

a. Establishment of impermeable slabs.

With the construction of the impermeable slabs and drains the water filtering under the sheet piling could flow through the drains without carrying the fine particles with it and without provoking ground settlement.

The construction of the drainage had to have such an effect that the upward pressure of the water on the undersurface of the slab, i.e. at 263.2 m, was balanced by the weight of the slab itself. In Fig. 7, the distribution of water pressure under the consolidated slab is indicated in respect of varying numbers of drains installed in the experimental model.

If one assumes for the specific weight of the consolidated zone a value of γ = 2.2, and supposing the slab to be 1 m thick, then the ideal level of the water under the slab which is under tension as in an artesian layer, should never exceed 263.2 + 1 x 2.2 = 265.4 .. (1), and in addition, a coefficient of safety must be observed. The method of calculation was as follows: by experiment, a suitable draining system was established by which through equal productivity of each well the levels of all could be maintained at about the same height.

As the uppermost layer had a relatively high permeability the water level outside the sheet piling was only slightly lowered. The calculation was made approximately as if the foundation trench was surrounded by an unrestricted water level. In these calculations, for every actual well, a further imaginary well was placed symmetrically on the exterior of the sheet piling and it was assumed that the same quantity of water was pumped into these imaginary wells as was yielded by the actual wells. Thereby the water level against the sheet piling was held at its original height as was approximately actually the case (See diagram 1 and 2 below):

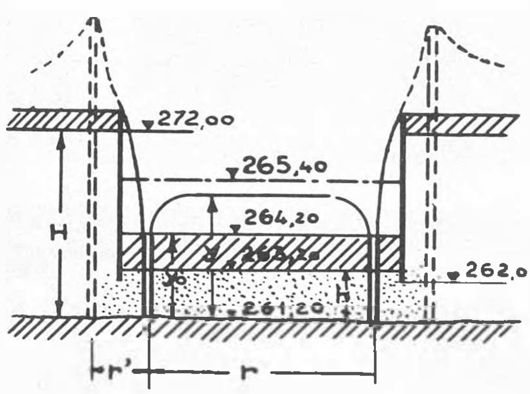


Diagram 1

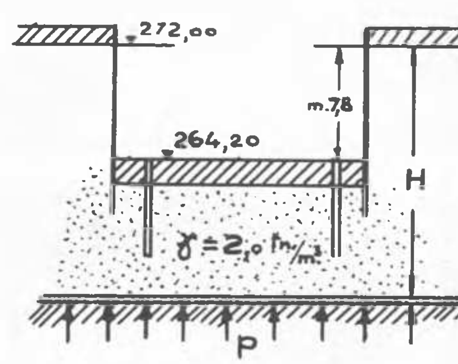


Diagram 2

Taking the following as averages of the actual values:

$$H = 272.0 - 261.2 = 10.8 \text{ m.}$$
$$y^{o} = 264.2 - 261.2 = 3.0 \text{ m.}$$
$$h = 263.2 - 261.2 = 2.0 \text{ m.}$$
$$K^{10} = 17.0 \times 10^{-4} \text{ cm/sec.}$$

the total quantity of water in the foundation trench (Q) was calculated to amount to 15 lit/sec. (Q = 15 lit/sec).

As the bottom of the excavation was on a level of 267 m. and the total infiltration was 7 - 8 lit/sec, such a value is quite possible.

At two points situated in the middle of the foundation trench the height of the water (y) was reckoned and for point N the calculation was:

$$N: \quad y = 3.6 \text{ m} \quad \text{or } 261.2 + 3.6 = 264.8 \text{ m}$$

and for point

$$M: \quad y = 3.8 \text{ m} \quad \text{or } 261.2 + 3.8 = 265.0 \text{ m}$$

For this second point the degree of safety (n), according to the equation (1), on this page, is as follows:

$$n = \frac{2.2}{1.8} = 1.2 \quad \text{or } 20\%$$

Altogether 72 drain pipes were proposed and the setting up of Piezometric tubes within and with-

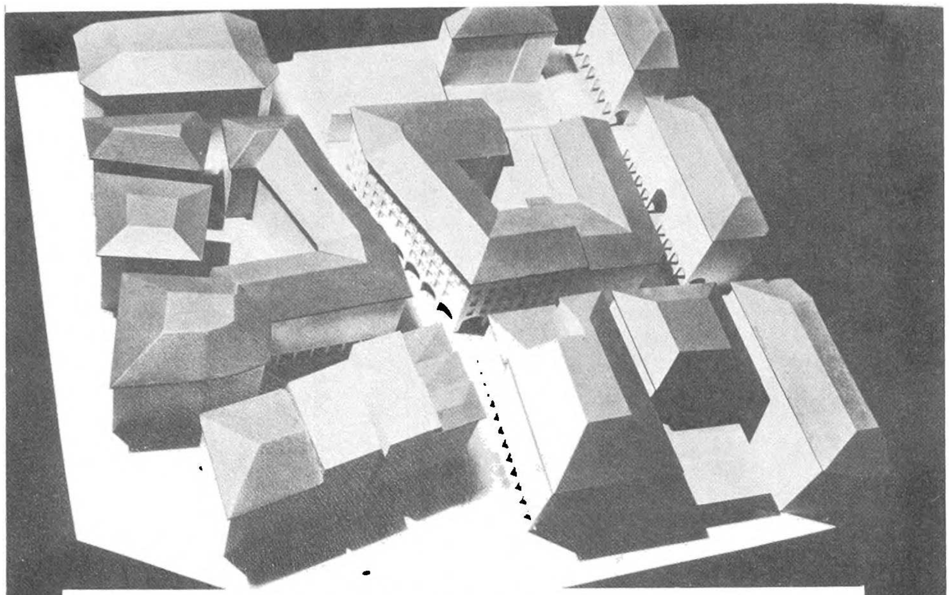


Fig. 1. Model of the Building Site and the surrounding Buildings.

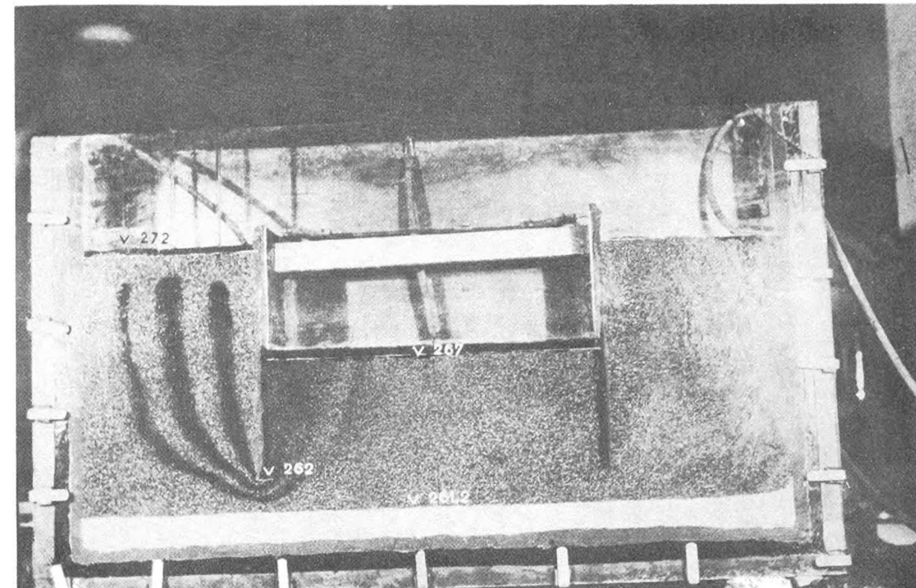


Fig. 6. Model of the Foundation Trench before consolidation work.

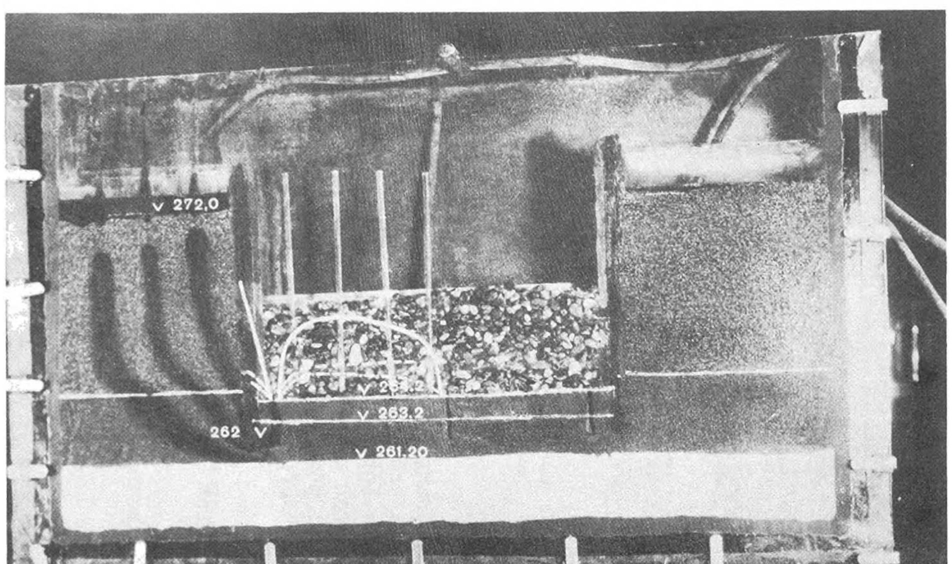


Fig. 7. Model of the Foundation Trench showing consolidation work by means of a consolidated slab (Proposal A).

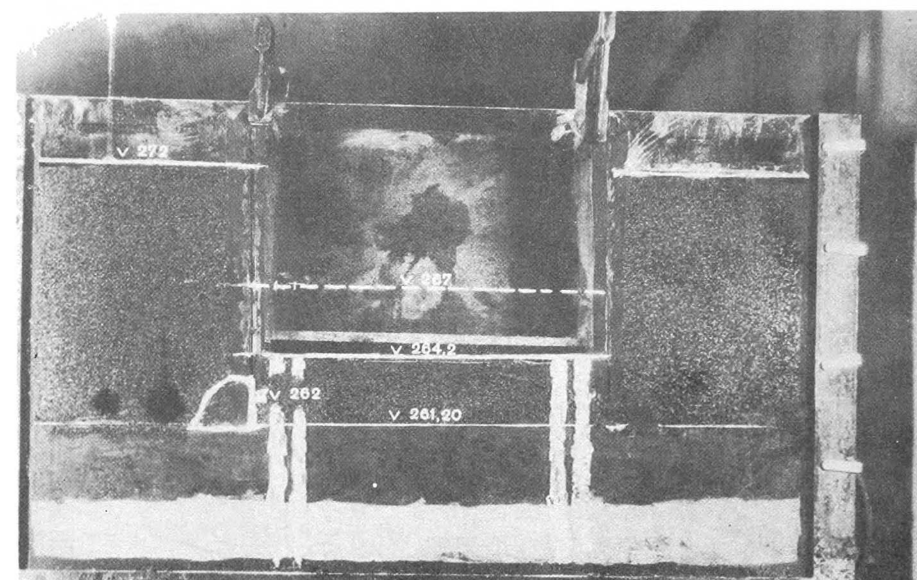


Fig. 8. Model of Foundation Trench showing consolidation work by means of extension of the sheet piling by chemical injection (Proposal B).

out the foundation trench for the checking of the drainage work was suggested so that observation of the water levels during the entire work could be made.

Now the question had to be solved as to how deep the drains were to be conducted. At this point, the following consideration was observed: through drainage the danger of erruption should be removed. There was, however, the possibility that very thin permeable, or worse still, very impermeable layers might run through the ground. These layers can be a few cms thick and can be observed in certain places when boring and when inserting the piles.

In one such case (Diagram 2) the entire pressure of the column of water (H) might be directed on one of these layers so that an erruption would be caused even if the water pressure on the chemically consolidated slab was completely absent. The drains were, therefore, so deeply conducted that such a case could safely be prevented. Their required depth was reckoned as follows (Diagram 2):

$$H \leqq \gamma \times (H - 7.8) \text{ and hence}$$

$$H \leqq \frac{7.8 \times \gamma}{\gamma - 1} = 7.8 \times 2.0 = 15.6 \text{ m}$$

The drains were, therefore, laid at least to the level of 272.0 - 15.6 = 256.4 m.

Then it was shown that the underground had in its deeper layers a large portion of organic materials which consequently caused a considerable evolution of gas (marsh gas). It was, therefore, advisable for the drains to reach somewhat deeper to about a 254 m level.

b. Execution of the Drainage plans.

On the basis of the curve resulting from tests on the soil and showing the size of the grains, the sizes of the grains of the two layers of sand filter in the drains were graded in such a manner that the outer layer consisted of grains sized 0.2 - 4.0 mm and the inner from 2.0 - 5.0 mm. In this way the danger of the action of washing away was eliminated with sufficient certainty.

In order to facilitate the work of injection it was necessary to place the drains in position prior to the injection of the chemical substances. Thus the earth would be more stable and the movement of the underground water reduced, i.e., it diminished on the one hand the danger of an erruption during the injection process itself and the pressure of the injection could, therefore, be kept relatively high; and on the other hand, the possibility of a washing away of the chemical solution.

The drain tube had, therefore, to be sunk in the customary manner to the level of 254 m and the drain itself be fixed to a level of approximately 262.5 m, and the tube was then drawn up to this level. The filter sand was filled in from the surface and the drain tube terminated at a level of 267 m. After the preparation of the injection zone, the drain tube was entirely taken away so that only the sand filling remained in the earth and, during the excavation it was taken away at the same time as the ordinary earth.

c. Extension of the sheet piling by means of the injection of chemicals.

To illustrate the 2d solution which later was actually followed, a further experiment (See Fig. 8) was carried out by means of a model. Beneath the level of 261.2 m, a layer of silt - soil from Lugano - was filled in as impermeable material. The lower edge of the sheet piling was fixed on a level of 262 m and the remaining part filled with coarse sand. The piles were outlined with chalk on the outside of the glass casing used for the model.

To prevent the lowering of the water level outside the foundation trench it was at first necessary to cut off the communication with the trench, i.e., the sheet piling had to be extended to the impermeable layer by means of the injection of chemicals. This zone of injection is indicated in Fig. 8 by the white area, but, in reality, it reached deeper into the trench so that it received support from the piles.

Then on the left half, immediately behind the injection zone, colored material was injected at two points in order to indicate a possible movement of water. Then the trench was "excavated" from a level of 267 m to one of 264.2 and the water in the trench drawn off. The model was left standing 24 hours without the water penetrating into the foundation trench and without a fall in the water level (standing at 272) outside the trench (Fig. 8).

In order to prevent an erruption at a greater depth (See diagram 2), the presence of vertical drains was also necessary, as had been already suggested by the Specialized Firm, and they should reach down to the level of 254 m. The work of inserting these drains was of a similar nature and kind as was foreseen in the first proposal, only that the number was much decreased as it only concerned the releasing of the tension, and not the creation of an efflux of a nominal amount of water. As in the first proposal, the presence of Piezometric tubes and the observations of the water level therein gave the necessary data for the calculation of the required number of drains.

Actually 24 drains with a total length of 256 m were installed.

3. Laboratory experiments regarding the selection and combination of the injected material.

a. Cement

For the injection into the larger gaps and spaces a standard cement was used which had given good results on similar soil on many other occasions. In this particular case one had only to concentrate on the time of setting, the fineness of the grains and the resistant powers.

b. Chemical substances.

When the soil to be treated consisted of such small particles that the fine suspended grains of

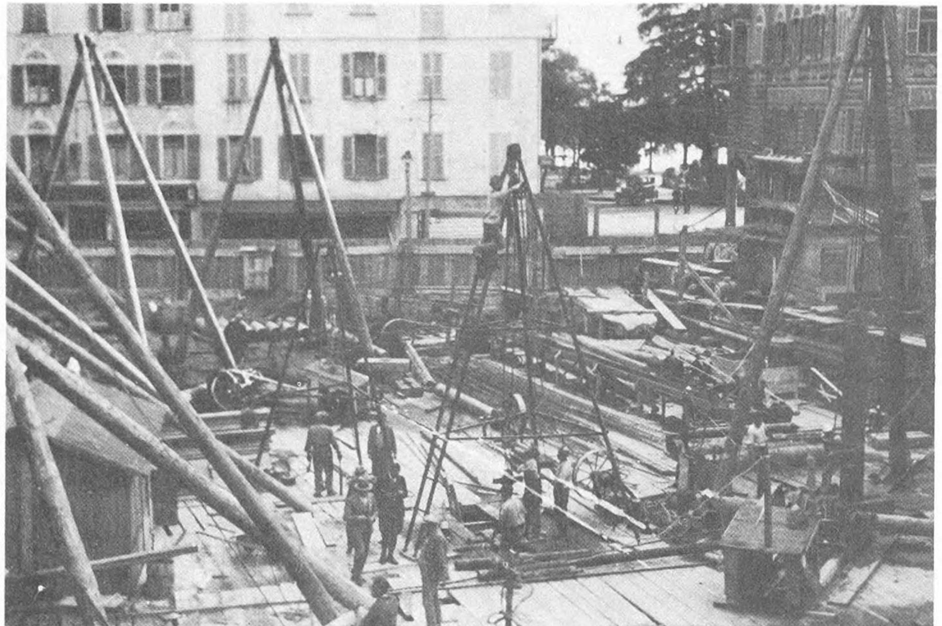


Fig. 9. Pile Boring Work in Progress.

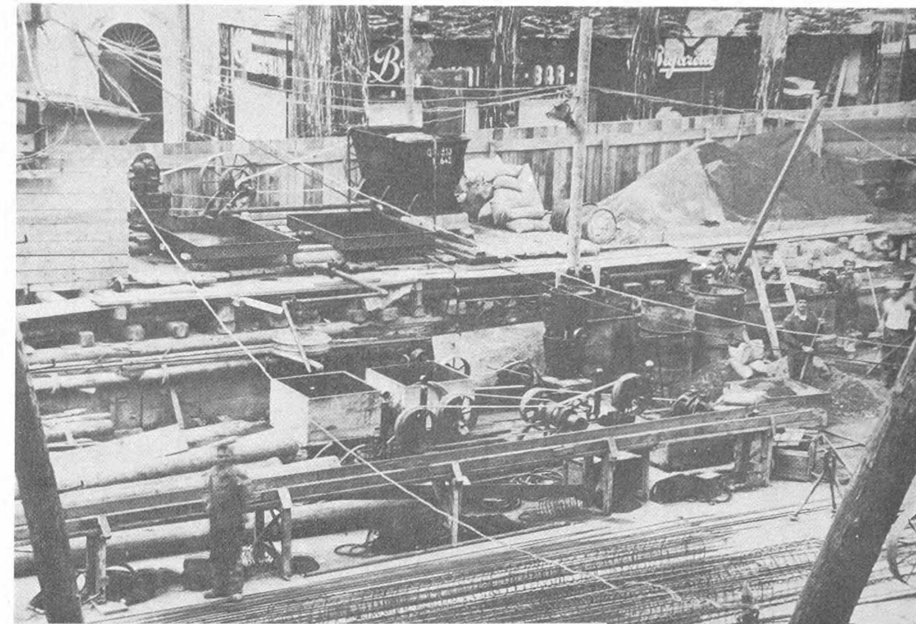


Fig. 10. Installation of Injection Apparatus.

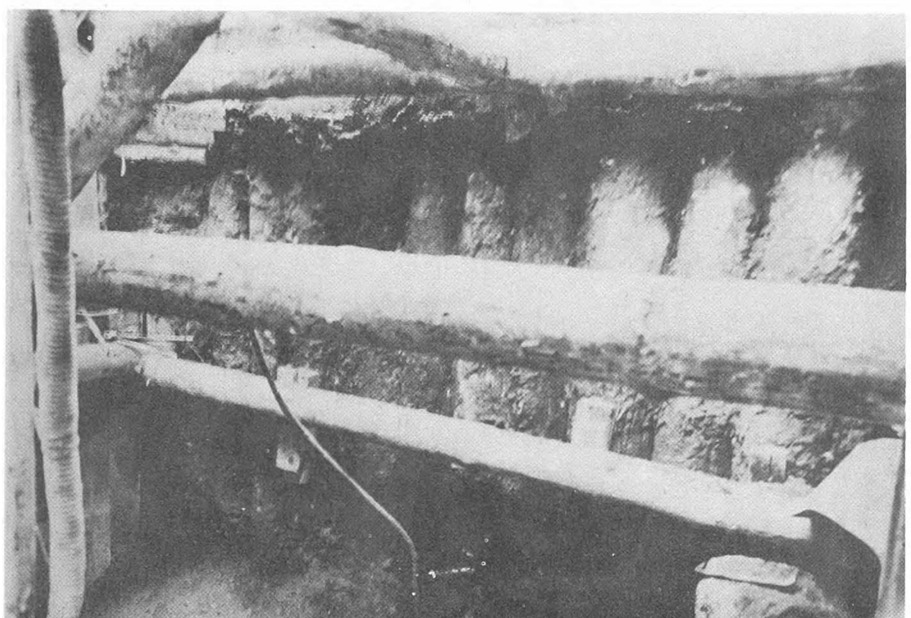


Fig. 12. Sheet Piling by means of Rodio Piles.

Fig. 13. Excavation of the Foundation Trench to a level of 264.2 m., showing heads of Rodio Piles.

cement could not penetrate further, it was necessary to use chemical solutions for the creation of impermeability and resistance.

"Rodio Gel" was adopted for injection, it being a product of the basic reaction of a Calcium compound. The two necessary ingredients, A and B, were mixed on the building site and the resultant, slightly viscous, completely homogenous fluid was pressed into the ground through one injection tube.

As the Gel is only slightly viscous, a proportionally light pressure was sufficient for a large radius to be covered. There is also a further and greater advantage in the use of this substance in that the commencement of "setting" can be regulated by taking into consideration the conditions of the ground, the flow of underground water, temperature of the earth, etc., and therefore the Gel can remain liquid during the entire process of injection. Thus, the outer surface of the injected substance does not prevent, by premature setting, the inflow of further quantities.

The penetration of a single zone can also be achieved by injection through several tubes at the same time and a uniform setting and uniform impermeability of the treated layer can be assured. (This method was studied and perfected some years ago in the Firm's Laboratory in Milan and in Professor Terzaghi's Laboratory in Vienna, and was previously extensively used for the construction of a 50 m high rock filled dam in Algeria. In this latter case the concrete cut off wall was extended into the sand slopes of the valley by means of a chemically consolidated zone. Similarly its depth was also extended. The magnitude of this work is shown by the great cost which finally will probably reach three million dollars.

A smaller, but technically important application of the same method was made by the Specialist in the case of the consolidation of the leaning Tower of Pisa.)

Before the beginning of the work on the building site itself, experiments regarding the appropriate composition of the Gel in relation to the start and continuation of the setting process were made in the Laboratory. After testing different mixtures the time for setting was fixed at 3 - 6 min. and the beginning of the process at 45 min. to 1 hour after the injection had been made.

This rapid period for setting was in this special case very advantageous as in this manner, any possible shaking movement or washing away of the Gel during the delicate process of setting did not produce harmful results.

Other experiments were made to determine the resistance of the soil samples in relation to the distance of the point of injection.

One realized from the diagrams that, with an increasing distance up to 1 m from the point of injection, the consolidated material had practically the same resistance capacity, so that a uniform treatment was assured.

<u>Practical Execution of the Work of Injection.</u> After examination of these circumstances and after obtaining good results from a test injection in a large sized model in the Laboratory, the practical execution of the work could be proceeded with on the building site.

1. The injection of cement was used:
   a. To consolidate the particularly endangered Zone A;
   b. To support the dividing walls of the houses immediately touching the foundation trench;
   c. As preparation for the chemical injection beneath the sheet piling and to unify completely the pile wall;
   d. For the filling of the large gaps and spaces.

The satisfactory results of the work were immediately noticed. With continued injections, a considerable diminution of the settlement movements was observed and the latter, after a short time, entirely ceased.

2. Rodio Gel was successfully adopted in the following instances:

a. For the consolidation and impermeabilization of a section of the soil in Zone D, so that a well, to collect and distribute the water flowing from the horizontal drains under the concrete basement slab, could be established;

b. For the purpose of closing up the sheet piling (made from a series of Rodio piles, one touching the other--Fig. 12--where the strong stiffening of the foundation trench did not permit the introduction of piles;

c. For the introduction of the horizontal slab between the levels 263.2 and 264.2 m. The consolidation of this slab produced many practical difficulties, as on the one side the covering of only 3 m of earth was too little to prevent in the soil already disturbed, a break up during the injection process, and on the other side, owing to the arrangement of the drains round the foundation trench, a slight sinking of the underground water level on the exterior of the sheet piling was feared, and as a result, a noticeable sinking of the earth in certain places;

d. For the consolidation of the ground beneath the sheet piling reaching to the practically impermeable layer (See Fig. 3 and 4).

The work of injection proceeded without any remarkable difficulties. However, there were occasional erruptions when the covering earth (consisting of sand) did not suffice as a counter-weight against the pressure of injection. Nevertheless, by various ways, for example additional weight by means of sandbags on the endangered zone, repeated injections, etc., such difficulties could be overcome. The necessary injection pressure differed according to the quality of the soil and amounted to between 2 - 8 atm. The installation of the injection apparatus is shown in Fig. 10.

The closing of a 2 x 1.8 m window, which had been left during the construction of the iron sheet piling, proved to be particularly delicate. A large quantity of water, rich in sand and other fine

DIAGRAM SHOWING TESTS ON BEARING CAPACITY OF PILES

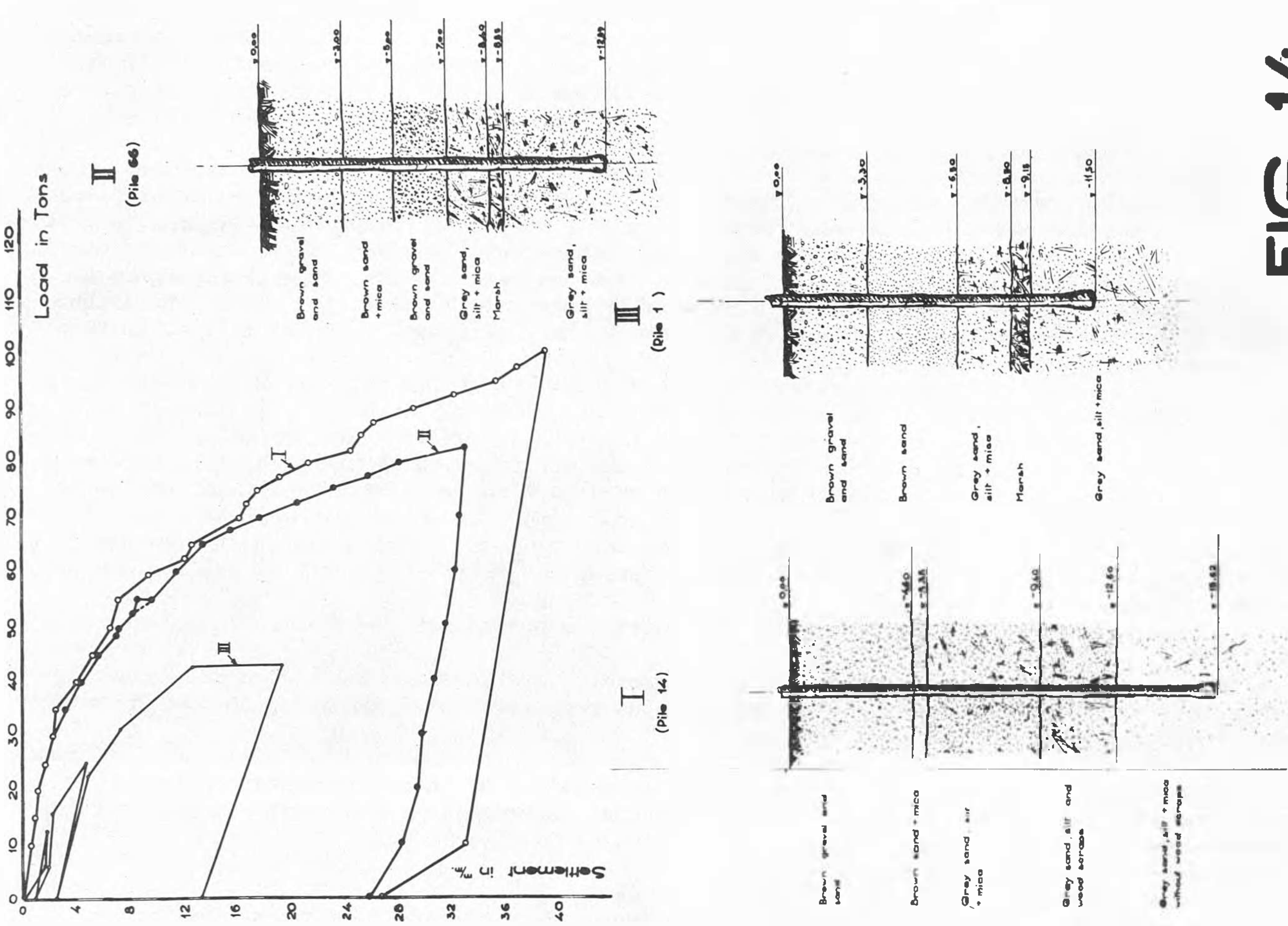


FIG. 14

INFILTRATION OF WATER INTO THE FOUNDATION TRENCH DURING THE INJECTION PROCESS

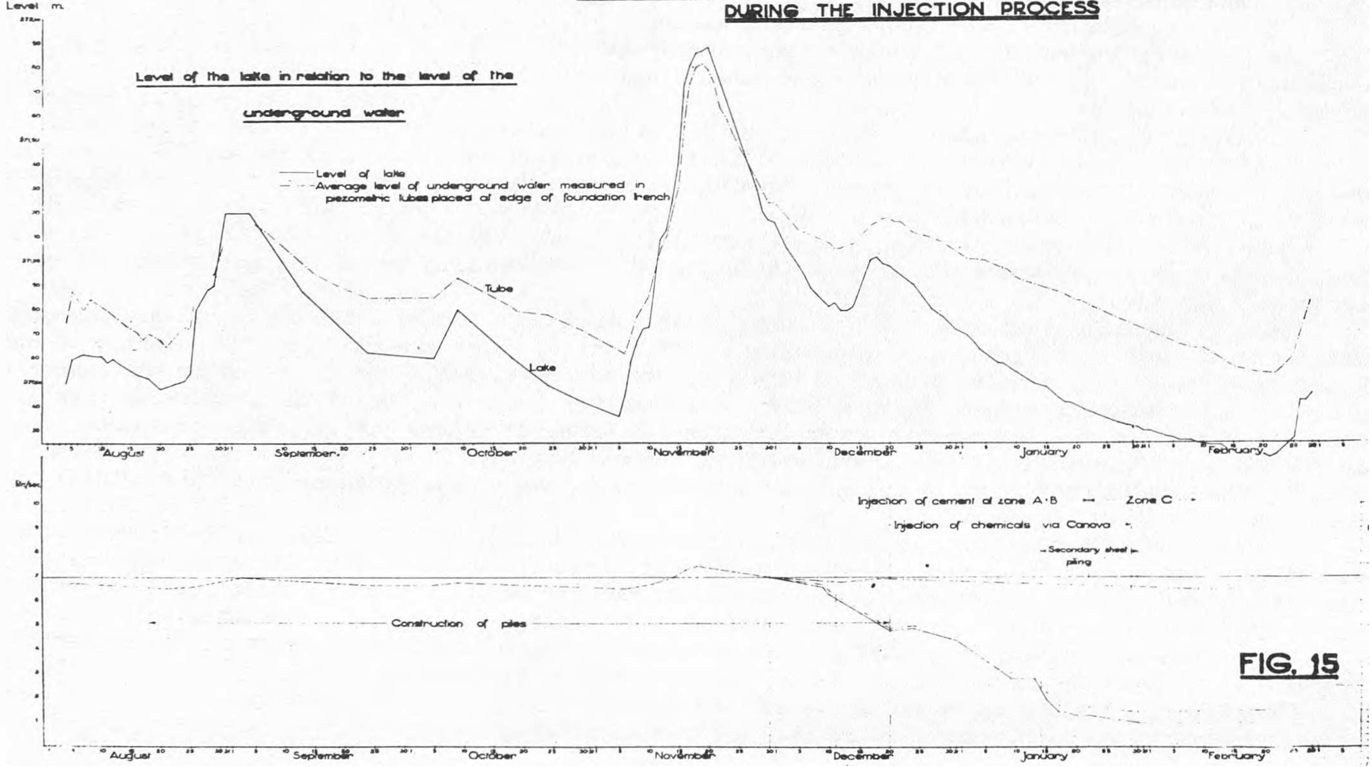


FIG. 15

particles, streamed through this window. At this point the composition of the Gel had to be so chosen that instantaneous reaction and setting took place. Thus, in a short time, the influx of water was reduced from approximately 0.5 lit/sec to practically nil.

The width and depth of the injection zones were adapted to the course of the sheet piling in relation to the impermeable stratas.

The well in Zone D was surrounded with a 1 m thick consolidated wall and a $1\frac{1}{2}$ m thick floor.

Where the end of the sheet piling was much higher than the impermeable layer, the height of the consolidated zone of 1 - 5 m was foreseen with a width of 3 m, two of which lay within and 1 m lay without the sheet piling; for the remaining part, a width of 1 m on the outside of the sheet piling was sufficient (See Fig. 3 and 4).

Results.

1. Piles. The construction of the Rodio piles could be carried through without the slightest disturbance of the most endangered and unstable building ground and its surroundings. Special preventive measures were taken in order to have sufficient counter pressure in the boring tubes during the boring process and to avoid an upheaval of the ground at the termination of the boring tubes.(The Rodio boring piles necessitated for the work of Lugano the boring with tubes of 35 and 45 cm diameter but in special cases also the diameter of 60 and 100 cm can be used. When the tube reaches the desired depth and the material therein is excavated, concrete is introduced in various manners according to the length of the pile and its diameter. With Rodio bored piles, the boring tube is generally extracted with the help of hydraulic pressure after the insertion of the concrete. The base of the piles in this system is also generally injected.)

The pile wall between Zone A and B (Fig. 12) which was fitted up in the shortest time with the help of two boring apparatuses, could in conjunction with the injection of cement, completely suppress the movement of the specially endangered Zone A.

The bearing capacities of the piles were calculated on the basis of soil mechanics analysis and, by means of load testing with three piles of different lengths, were actually checked on the building site itself. The necessary coefficient of safety for the working load was, therefore, guaranted (Fig. 14). The relatively small bearing capacity of Pile III was attributed to the presence of a 0.35 cm thick and soft marsh layer near the upper end of the pile. This layer, capable of large compression, permitted a rise of the ground around the foot of the pile, and it was therefore, decided to drive the piles deeper when similar conditions in the ground were present (See Pile I). However, it was later shown that marsh layers of such an unfavorable nature were infrequent.

On account of this test the average length of the piles was fixed at approximately 12 m and the working load at 40 - 45 tons.

The total production amounted to 275 bearing piles (diameter 45 cms) with a total length of 3500 m and 175 sq m of pile wall, with an average width of 35 cms. (A French Firm executed shortly a very important foundation for the Nice Gas Works which required such piles of 36 m. These piles were cast by means of a concrete pump. The same method will be applied by an Italian Firm for the foundation of two bridges at Pisa which will require some piles of more than 40 m length.)

2. Injection of cement. The results of this were satisfactory where it was necessary to fill the gaps and spaces and to prepare for the chemical injections, especially in Zone A and under the houses touching the foundation trench and to effect a complete unification of the pile wall.

Altogether 388 tons of cement were injected through 165 tubes which had an average length of 11 m.

3. Injection of chemicals. The satisfactory result of the chemical treatment of the soil showed itself very soon. In Fig. 15, the diminution is indicated of the influx of water as work proceeded. While before the beginning of work, the water influx into the foundation trench amounted to 7 - 8 lit/sec, a noticeable diminution was observed a few days after start of the work of injection, and the quantity of water to be drained away after the finish of the injection round Zone B amounted to a total of merely 1 lit/sec.

The experiments, which are indicated in the figure, were not made until the completion of the foundation work, but were finished after the completion of Zone B. After the beginning of work in Zone C, the General Contractor immediately began the excavating of Zone B. Filter wells and drains were laid down so that a further exact checking of the inflow of water was impossible. In any case, it can be assumed that the quantity of water which, before the start of the impermeabilization work in Zonoe C, amounted to 1 lit/sec was substantially reduced after the completion of the work, especially as some localized infiltration points completely dried up as the work proceeded.

The practical results of the impermeabilization work were finally apparent as the excavation work was continued. The prescribed levels could be attained everywhere without pronounced difficulties and without the erruption of water or fine sand (Fig. 13). In a few places, water filtered in, but in such small quantities that the excavation work was in no way disturbed (Fig. 11).

Altogether 1665 cu m of Gel were injected through 470 tubes of an average length of 10.5 m.

During the excavation the neighbouring buildings were repeatedly surveyed which certified that all movements had ceased and the stability of the newly created equilibrium was assured.

The laying of the foundation slab and the basement floor, as also the erection of the aboveground building, proceeded rapidly and surely so that exactly one year after the completion of the consolidation work the coping could be placed on the roof of "La Baloise" building.

No. N-14

OPENING DISCUSSION

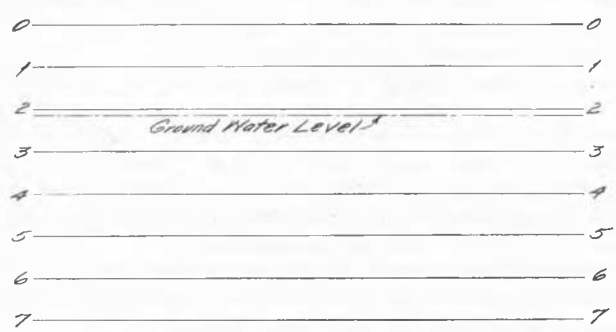
Lazarus White, Consulting Engineer, Spencer, White & Prentis, New York City

Annually I give a few lectures here at Harvard on the subject of foundations and for each lecture I am allowed two hours. I find that too short. This time I get twenty minutes to cover the whole subject. As I understand it, the principal function I have is to start a discussion. This is the place, I understand, where real discussion starts, and naturally there will be differences of opinion. There are plenty of differences of opinion in this country in the various localities, and when we get the whole world we will probably find larger differences of opinions.

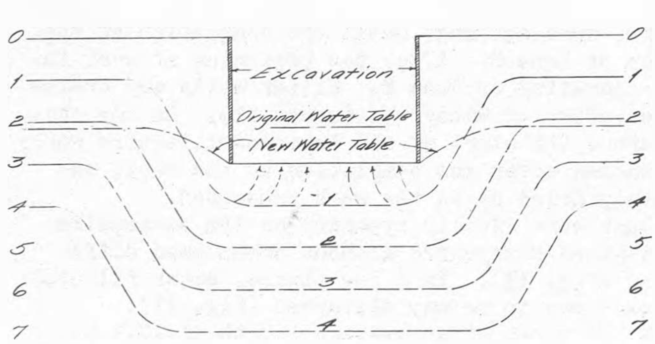
I was much interested in Dr. Terzaghi's remarks this morning in which, with his sharp scimitar, he cleared away a lot of dead wood, a lot of growths which have encumbered this subject for a long time. It used to be, when I graduated, that we had a few very simple rules. All we had to know was the value of clay, sand, and so forth, in tons per square foot, and how much a pile would stand. It either stood fifteen or twenty tons. You applied a certain formula and there you were. At the most you poked a few holes in the ground by means of a wash drill and watched the contents in a pail, and there you were.

Now unfortunately, some of the older engineers think that the subject has become very complicated and you can no longer apply simple rules. The proper way to express it, perhaps, is that you can no longer apply rules of thumb. If this subject could be handled by a few simple rules, as I cited, there would be no use for this conference and there would be no use for engineers on these problems.

The best analogy that I can make for an engineer nowadays, when each foundation problem is a separate one, is that he is somewhat in the capacity of a physician. He first has to diagnose the case. He has to find the conditions of the problem, particularly in the nature of supporting underground. To do that he can't do it so easily as a doctor can, who can stick a man against a screen and X-ray him. He has to look deep into the ground. The more we go into the subject the deeper into

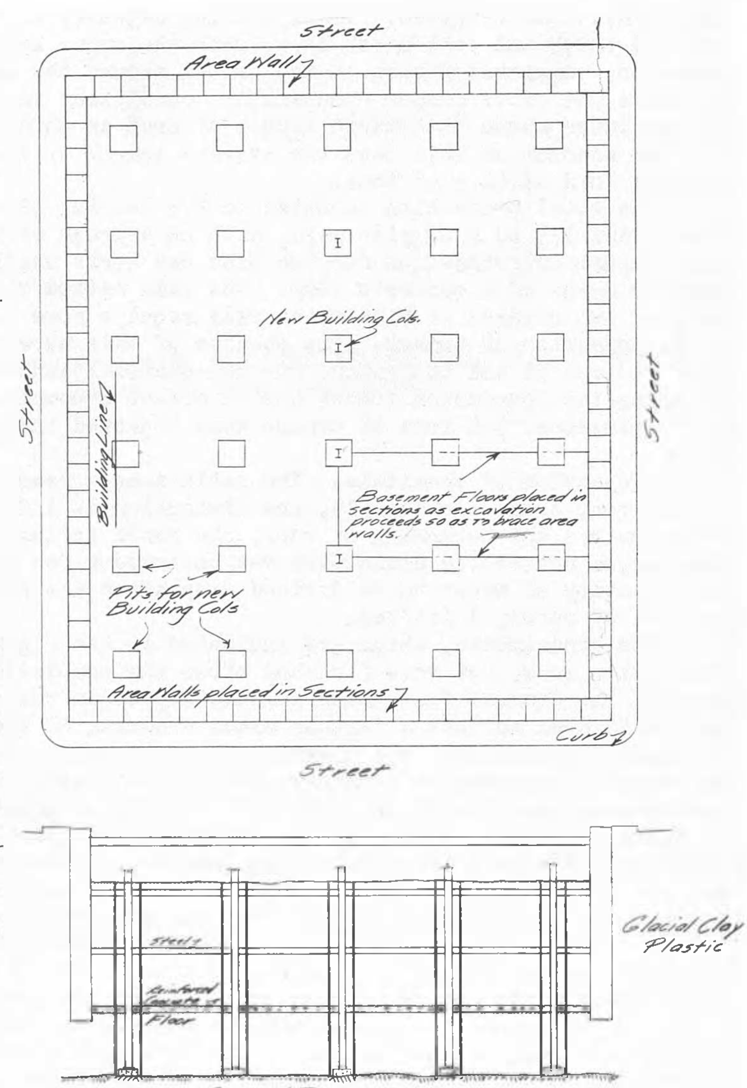


Lines of Equal Vertical Pressure in level plain of indefinite extent.
Fig. 1



Distribution of Lines of Equal Vertical Pressure by excavation for building. Also showing distortion of Water Table, creation of upward flow of water, and quick condition.

Fig. 2



Method of Excavating and Placing Building in Clay. Great Lakes Region.
Fig. 3

the ground we have to go. I remember that there used to be some arbitrary rules that the settlement occurred in the first four feet below the bottom of the footing. Now we know that the seat of the trouble may be at great depths--hundreds of feet. All in all, there is a difficult problem there, and I can't see why engineers should regret the value of their profession.

The subject is illustrated rather clearly, I think, by this fact. Once upon a time the American Society of Civil Engineers had a "Committee on safe bearing value of soils for foundations". This committee was to put opposite each name, like clay, sand, and so forth, a value -- 4, 5, 6, 7. The engineers were to use that for greater safety. Well, if they were able to do that it would completely abolish the foundation engineer. This committee worked for several years and handed in reports. Then they gave it up in despair.

Then it was that I was called in to resume this work as the chairman of a new committee whose name is Committee on Earths and Foundations, which is very different from the original name and shows the nature of the problem. It is a problem of soils and foundations. By calling it earths and foundations you bring in the geological problem. Whereas a physician has to know a patient's past history, we have to know something of the capacity of the ground on which we are working, whether it is a recent formation, a pond deposit, a river deposit, or what not. Therefore an engineer in foundation problems should be something of a geologist, and the better geologist he is the better foundation engineer he makes, in my opinion.

On the theory of foundation I just want to mention some of the papers bearing on this problem, which I was required to review.

Here is a plane of approximately level layers over a wide area. (Fig. 1). These are relatively soft deposits. Now the pressure, of course, increases uniformly. Every foundation starts with a hole in the ground, you might say. The size of that hole is extremely important in this problem. If it is a hole ten by ten it is one problem. If it is a hole three hundred by three hundred, for a large building, it is an entirely different problem. Here is our hole (Fig. 2.) We dig down in here. What happens? Referring to Fig. 2 we must connect the lines of equal vertical pressure outside the foundation excavation with the corresponding lines below the excavation, which necessarily means distortion of the pressure planes and high stresses in the corners--on the whole a violent change of stress conditions.

Also there are abrupt changes in the hydraulic conditions; the water table is distorted and differences of head created causing upward flows at the bottom which may render the bottom "quick"; also, if impervious layers exist, such as silt deposits, the bottom may be put under hydrostatic or artesian pressure. This may be the cause of the rise in the bottom at the Lottery Building in the City of Mexico, and the failure of the sides--the bottom behaving as a beam, and the sides as abutments. In addition to that there are changes in temperature. The surface underground comes in contact with the air and dries out along those surfaces.

You can't dig a hole instantly and stick a building into it. The science of foundation building hasn't quite approached that point. We must, in this field--if I seem to be a harsh critic it is not intentional--beware of falling back into some of the same errors that we had before--over-simplifying this problem. Therefore I do not believe that all you have to do is to push a building into the ground and you won't have any more trouble if perchance you can make the weight of the building equal to the weight of the material in the hole.

One of the papers describes a very skilful job of putting in a stiff foundation of trusses on a comparatively small area and putting a building on it. It was a very scientific foundation. They expected settlements and got about what they expected. You might fairly say, "Well, what would you do with this problem?" It is a proper question.

All I can say is that we have had that problem and we have done something about it. In Chicago, and in the Great Lakes district, there are large deposits of plastic clay where these problems arise. It is not nearly as bad as glacial conditions, but it is still bad. At the beginning--you know contractors used standard methods, tried to brace their cuts and got into a lot of trouble and had collapses. Then a method of excavating in clay must have been devised by a man whose instinct gave him the right answer. They developed the method of installing a foundation with the minimum disturbance of original conditions. Here is a building (Fig. 3). It might be a square building, a large one. This represents a boundary wall on all four sides. There are streets here. This is clay deposit. Instead of digging out that whole area down to this and then installing the building, they merely dig sections of trenches and install part of the wall in there until this wall grows to enclose the whole area. At the same time they sink caissons, open wells, with as little loss of ground as they can. Then, before this is dug out, they put in the steel columns from the bottom of the pit all the way up. Then you have all the connections. Then they install the bracing through this wall. Then they dig a little of the clay out. The permanent steel of the building goes in and they go down floor after floor in that way, for perhaps six floors. There is the permanent bracing, the floors of the building incorporated in these walls that serve as rigid bracing, floor after floor.

Now, when you get down you have the hole excavated. Then the bottom. They are very careful about the bottom. They excavated taking out the bottom in sections. In that way you have a building pushed into the ground, you might say. You have disturbed the material only the minimum amount. That, I think, is a very good solution of that case. Others might be devised. That method is not too expensive a one, but it requires a good deal of skill.

So much for that. Now, as to pile foundations. I think Dr. Terzaghi has covered that field pretty well. There are an infinite variety of pile foundations which may be devised. Each variety has a good advertising value. But to me every foundation fundamentally comes back to a spread footing. Some way or other there must be below that building a layer which takes the entire weight of the building and the soil, water and everything above it. No matter how much you puncture that upper strata with piles that layer must do its work. How does a foundation do its work? That is the basic problem of soil mechanics which we have to solve.

No. N-15 DISCUSSION ON THE MOVEMENTS WITHIN FOUNDATION PITS DURING CONSTRUCTION

Participants: J. A. Cuevas (Mexico), Lazarus White (U.S.A.) and Karl v. Terzaghi (Austria)

J. A. Cuevas. The universal foundation practice consisting of spreading footings according to calculations based on the so-called "bearing value of soils" even when determined by small area tests made on the spot itself, has definitely and apparently failed in Mexico City, as far as large surcharged areas or heavy buildings are concerned. Bearing values as low as half a ton per square foot (.5 kg/cm^2) are by no means safe even for very light isolated houses, where the settlement may not become apparent in comparison with the surrounding ground: but discloses itself in a long course of years--when it becomes necessary to raise the old sunken sewers.

Evidently calculations and foundation methods shall be based on soil stress-strain relations, but lack of reliable laboratory data among other reasons, was the main determining factor to make an actual size experiment in the new Lottery building designed to have 17 stories above the ground and to replace old buildings two stories high which because of its great and uneven settlement was no longer useful. The new building being, moreover, irregular in shape and its weight very far from being uniformly distributed on the loaded area, it was necessary to spare neither effort nor care to solve the problem. Then it was resolved to take advantage of the job: (1) to make positive investigations as to get a basis for a logical and safe solution for the present case; (2) to derive general information as to the behavior of the subsoil of the City.

After removal of old buildings and having removed more than 14000 tons of earth from the excavation the uplift recorded for the central part of the bottom of it, was more than 4 feet (127 cm). This maximum rise surpassed all expectations and as the upheaval of the bottom was not uniform, the flexible sole described in Paper No. N-5, Vol I was designed. In order to let this sole down to its final position of equilibrium before carrying up the structure itself, the rigid substructure is being ballasted so as to duplicate the pressures determined by the earth removed: ballast may be slightly increased in order to quicken the required presettlement, equal but opposite to the upheaval determined from levelling records of "testigos" (subsoil reference points or bench marks). We even foresee disturbances of levels due to the refilling of surrounding trenches, to the gradual and final restoration of the normal ground water level and to the mechanical hysteresis of the soil: to check

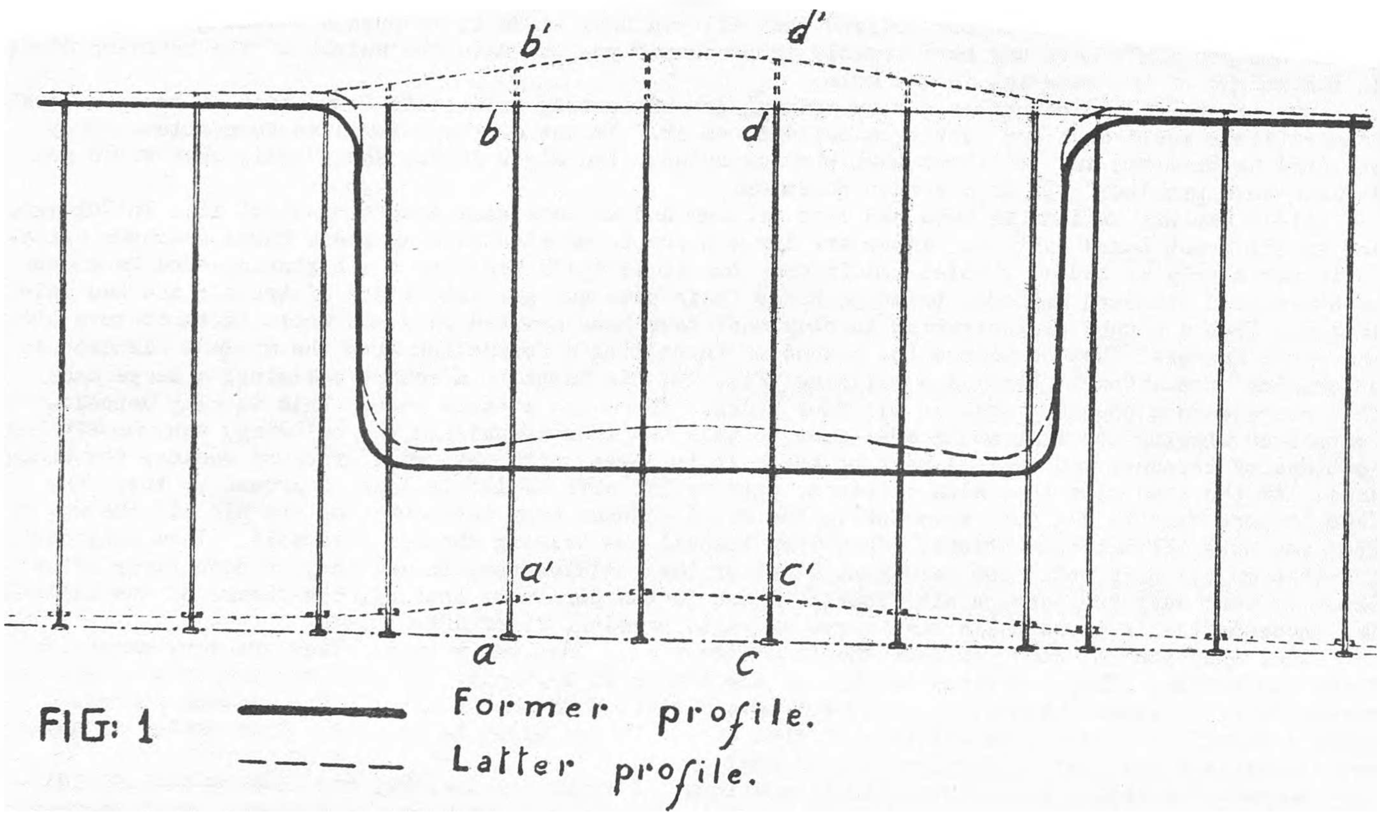


FIG: 1

these and other less important disturbing factors and to control the position of equilibrium obtained, extra excavation and extra ballast,--to be conveniently handled during and after the erection of the structure--have been provided in the design.

As to results of general importance which the use of "testigos" made clear let us mention the following:

1. By carrying up new buildings the settlement not only of the loaded area but of the surrounding ground also was made plain, without shearing failures being observed.

2. By excavating, it has been verified that not only the bottom but the borders as well, rose as illustrated in Fig. 1 without indications of shearing failure.

3. "Testigos" lined up vertically, with their shoes anchored at different depths made it clear the successive propagation of ascents at increasing depths in the course of time.

4. The weight of earth excavated and removed from the excavation, the varying form and dimensions --diameter and depth of hole excavated being especially considered--clearly appeared to influence deformations.

5. When by demolition of an old settled building its weight W is gradually diminished there is a certain fraction of it, αW, which will stand still, so that αW may be considered as a measure of the effective consolidation of the soil--other conditions being unchanged--Not to alter this equilibrium, the weight of a new building must be equal to the weight of excavated earth plus αW, when no final settlement is acceptable, on account, for instance, of sewerage strict requirements, as we ought to have in this almost level fill.

6. Comparing the load-descent graph of an actual size experiment with a laboratory stress-strain graph illustrating a normal single grained soil compression test (See Fig. 2) we find that segment AB would correspond to the settling of subsoil during the erection of a preceding building: segment BC to the removal of it; segment CD to the settlement of the new building until it reaches - at D - the same weight of building removed; finally DE would allude to the settlement due to the increasing weight of the new building until it is finished. When unloading or loading is discontinued, ascents or descents continue in an asymptotical form, the asymptote not always being a horizontal line (Fig. 3)

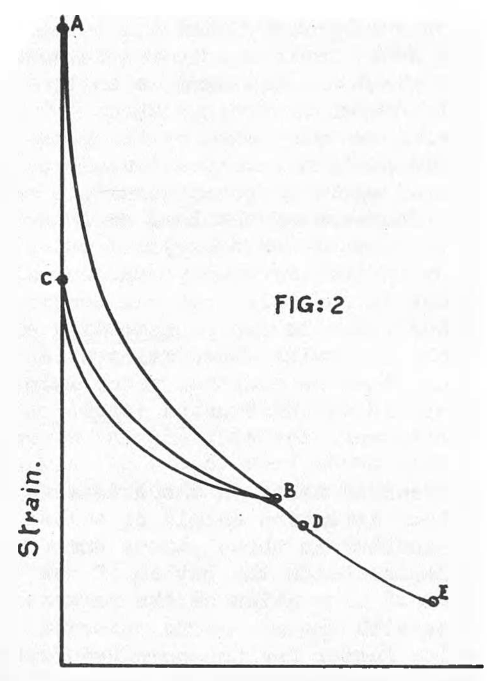


FIG: 2

The real difference between the building load-descent graph and Fig. 2 graph, as far as our experiment is concerned, lies in the fact that it is not feasible in actual practice to load or unload the soil with sufficient promptness to isolate the immediate effect from the mediate one; in other words: during the removal of old buildings and of earth excavated and while the new structure is being carried up, mechanical expansion and compression necessarily interfere with frictional and hydrodynamic lags.

7. On account of pumping done to have the excavation empty, the free level of ground water falls as illustrated in Fig. 4. The contraction of the dried area (marked hatched in the drawing) is very big when water content is as large as 3.21 and that is why the drainage of such area determines very noticeable settlements close around the excavation.

I agree that foundation conditions in Mexico City are very severe and contribute to extraordinary magnification of the subsoil movements; but I have heard that in Chicago somewhat similar conditions could be encountered and so I am asking Mr. White to inform the audience if he didn't detect any upheaval of the bottom of the excavation and any movements of the borders of the same when using the method he described before.

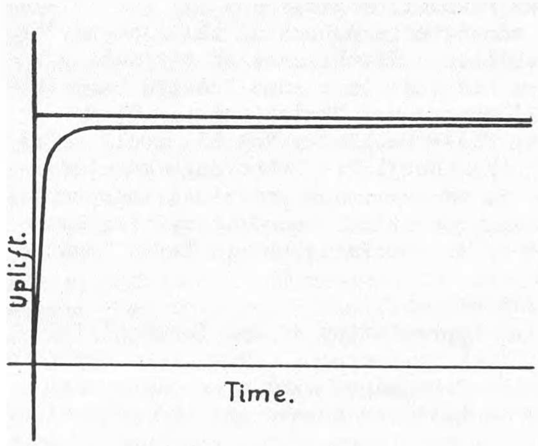


FIG: 3

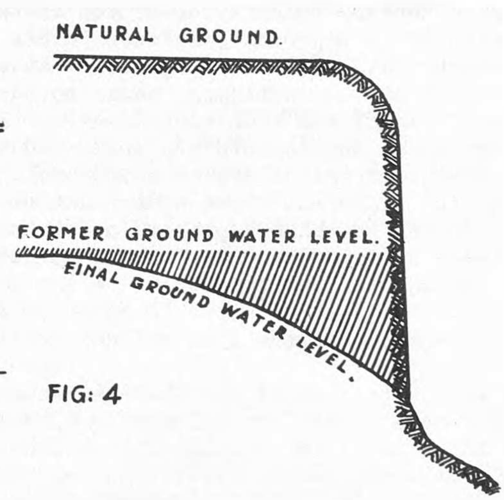


FIG: 4

<u>Lazarus White</u>: Mexico is an old country and we are a new country, yet Chicago deposits are considerably older geologically. Mexican lake deposits are very young compared to these glacial clay deposits in Chicago.

My answer is "yes". They do prevent these upheavals. It may be much more difficult in Mexico, but I think some of the methods may be applied there. Therefore I gave it for what it is worth.

I might call attention to the fact that when they do get down they remove small areas of the clay and replace it by reinforced concrete floor which is tied into the walls and the footings to prevent upheaval.

Karl v. Terzaghi: The work described in the paper by Mr. Cuevas impressed me as a remarkably bold and original piece of engineering. One of the most interesting features of the paper consists in the description of the energetic upward movement of the bottom of the foundation pit. Some ten years ago a pit was excavated for the basement of a sewage pumping station close to the Atlantic coast in New England. The pit had a depth of 30 feet and it covered an area of about 120 feet by 70 feet. To a depth of 14 feet below the bottom of the pit the ground consisted of a very fine, water-bearing sand which covered the surface of a thick bed of glacial clay whose water content was close to the liquid limit. In order to drain the pit the ground water level was lowered through a distance of about 14 feet by pumping from well points. Mr. L. White acted as a contractor and I myself was called in by the consulting engineer, Mr. Knowles of Pittsburg. Considering the high water-content of the clay every one of us feared that the process of excavation might cause a rise of the bottom of the pit which in turn would interfere with construction operations. In order to obtain reliable information as to whether or not such a danger existed, we established an underground reference point, similar to the "testigos" of Mr. Cuevas, located a short distance above the top of the bed of clay. The movements of this point have been observed during the entire process of excavation. To our surprise these movements were too insignificant to be detected with an ordinary level.

In connection with this observation the question arises as to whether the state of equilibrium in the clay was temporary or permanent. According to my opinion it was temporary, that is, I believe that the heave of the bottom was merely delayed. This opinion seems to be sustained by the following observation. A few years ago, after the construction of the pumping plant was finished, a boiler-house was constructed within a short distance of the former pit. It covered an area of about 100 feet by 70 feet. The foundation of the building consists of a concrete mat supported by the piles which did not penetrate the clay. The load which is carried by the foundation is approximately equal to 2.8 tons per sq ft. During construction and for a period of several weeks after construction was finished, practically no settlement was detected. Then, without any increase of the load or other apparent cause, the settlement started. Within a year it exceeded two inches and today, six years after construction was finished, the subsidence still continues. During the last few years several similar cases have come to my attention. As a consequence, I consider it possible that the compression or the swelling of a bed of clay can be delayed for reasons other than the low permeability of the material. Several days ago, Mr. Langer of Paris told me about the following observation which he made while experimenting with fat, Tertiary clays of the Paris basin. When he admitted water into the consolidation apparatus containing a thin layer of these fat clays in an undisturbed condition the swelling did not begin until about ten minutes after the contact between the clay and the water had been established.

One of the interesting features of the work described by Mr. Cuevas consists in the extensive use which was made of underground reference points. In this connection attention should be called to the Papers D-3 and N-3, Vol I, of the Proceedings. On the job described in these papers numerous reference points were established prior to excavation at different depths below the bottom of the river. On the basis of results of preliminary soil tests, the effect of excavation on the reference points was computed in advance. By comparing the theoretical results with the movements observed during excavation, it was possible to determine an empirical reduction factor for the computed settlement of the finished structure. By using this method a remarkably accurate estimate of the important and unequal settlement of the large structure was accomplished. The papers also contain the measures which were adopted in order to eliminate the excessive hydrostatic pressure which acted in the water content of several beds of sand located beneath the bottom of the foundation pit.

Another important function of underground reference points consists in detecting the seat of the settlement of existing structures in a state of progressive subsidence. In the case of a floating pile foundation an important part of the settlement seems to have its seat in a zone located beneath the points of the pile. (See Terzaghi and Fröhlich, Theorie der Setzung von Tonschichten. Wien 1936). The Shanghai buildings whose settlements are described in Paper No. F-12, Vol II, would offer an excellent opportunity to secure empirical evidence concerning this subject. A thorough knowledge of the seat of the settlement for different types of foundations is of paramount practical importance. I have more than one case on record in which expensive underpinning operations were without any beneficial effect, because they were designed to transfer the weight of the building onto a level located above the upper boundary of the seat of settlement.

J. A. Cuevas: I am heartily thankful for the kind and encouraging appreciation of Dr. Terzaghi for my paper.

As to the statement of Mr. White, let me say that the subsoil of Chicago being much older than ours in Mexico City, I readily accept that its deformations, due to building surcharges and excavation unloadings may have been so small as to be actually ignored, but they are unavoidable and must be of the same order of magnitude as the calculated settlements of buildings and should be detected by levelling proper subsoil reference marks ("testigos") referred to one or a few outside bench marks sufficiently far apart and so placed as not to be influenced by the displacements of surrounding ground and other accidental disturbing causes and so chosen as to fulfill the requirements of sufficiently precise levelling.

It may be worth while to try "testigos" to get information about the deformability of the undisturbed soil itself--as Dr. Terzaghi pointed out.--To do this, I would add, the unloading of subsoil from old buildings and excavated material affords a handy and almost inexpensive opportunity. These tests as a substitute to or complementary with local loading tests would also be useful to check calculations of predicted settlements.

Apart from general investigation on the behavior of subsoil when it is unloaded, the use of "testigos" (subsoil reference marks) has been very fruitful in Mexico City to explain the settling of light structures, even lighter than the earth removed--when built on the bottom of an excavation the upheaval of which had been disregarded, and also for ascertaining the disturbing action of the demolition of a heavy structure on neighbouring buildings as a consequence of the rebounding lift of unloaded soil.

R. Pietkowski (Poland): Did there, in the excessively difficult soil conditions of Mexico City ever arise a proposal of introducing the caisson foundation using compressed air and not disturbing the natural distribution of stresses in the ground? And if not, why not?

J. A. Cuevas: In the past, caissons were not used because it was considered very dangerous to destroy the three or four meters thick "crust" or topsoil which is stronger than the saturated subsoil. (The former is about 1.5 times heavier than water while the latter is only 1.2 or even 1.1).

Caissons were not used to build the foundation of the new Lottery building because it was the purpose to so conduct the job as to make the actual deformations of the subsoil evident and to measure those due to the removal of loads of the same order of magnitude as the weight of designed building: this case constitutes an actual size experiment on undisturbed soil for getting general knowledge as well as a positive individual solution.

As to the future, caissons of usual dimensions are not fit to build long horizontal tension and compression members as those of the substructure designed for the Lottery building.

Of course, for other cases, caissons undoubtedly may be used as much as other foundation methods in which the substructure is built without materially disturbing the subsoil pressures, earth to be excavated being employed as ballast which is gradually removed as the superstructure is carried up so as to always maintain the same subsoil stresses.

Karl v. Terzaghi: It would be interesting to learn how Mr. Cuevas prevented the underground reference points from gradually sinking into the soft ground.

J. A. Cuevas: Subground bench marks ("testigos") are anchored to the subsoil at the bottom of a vertical lined cylindrical hole, the weight of earth removed from which balances the weight of both the shoe and the rod of said bench mark. Allowance is made for the corresponding weight of water when the shoe is lower than the free level of it. The lining of the hole doesn't touch the shoe.

No. N-16

DISCUSSION

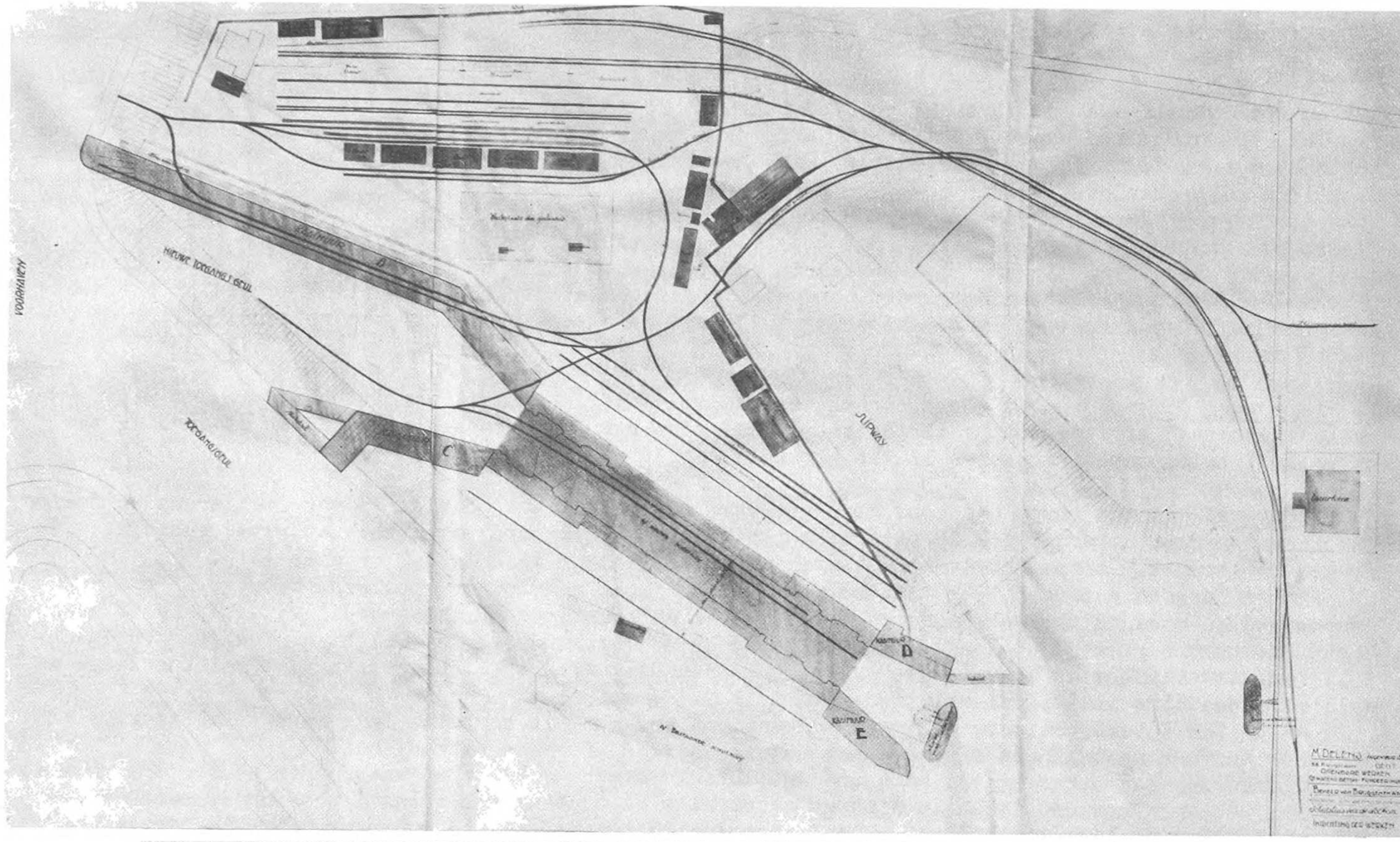
Paul E. Raes, I.C.C. University of Ghent, Ghent, Belgium

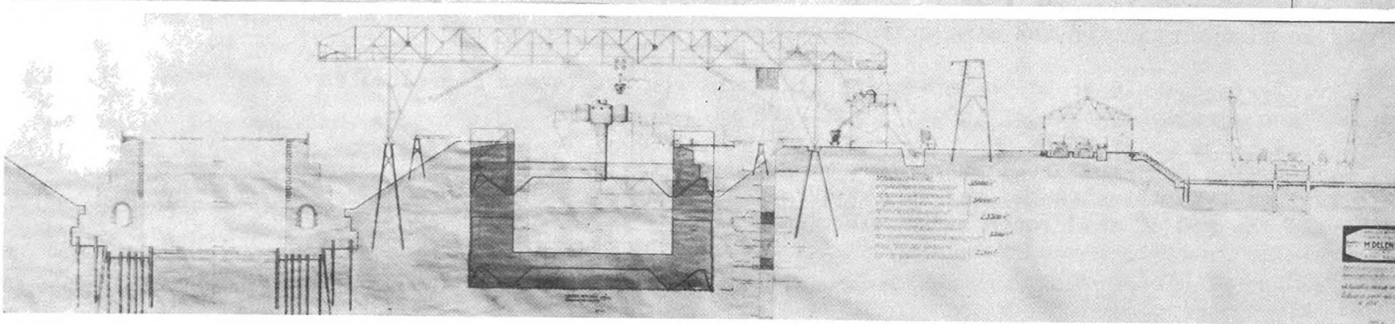
Yesterday I mentioned to a few members of the Conference a caisson used in Ostend and I noticed that it was not known here. I have presented a paper on lateral bearing capacity of piles and I will not talk about that now because people interested can read it in Volume I of the Proceedings, and if they are still interested after having read it, which is not always the case, they can have a full copy from me.

The caisson involves a new principle, and that is the main reason to talk about it in the Conference. We cannot go into details. We can only give principles, and this is an entirely new one and has even been patented in Germany, if I remember correctly. I have no written information about that caisson, so I must speak from memory and I am liable to make a few mistakes.

The idea is this. The human mind usually follows routine and the technical mind follows routine as well. That is, for instance, why the early motor cars were just the same as the horse drawn vehicle, and it has taken years to make out of a motor car what it is now--an air flow. The human mind cannot jump from one form, from one shape, to another. It must slowly develop. In the caisson business it is the same.

The caisson has always been looked upon as a simple tub which has been turned upside down. As in the tub, the sides and the walls are always on the exterior. It has never been imagined, up to that particular occasion, that the cutting edges of a caisson could be somewhere else than on the sides. In that instance in Ostend, a few years ago a lock was built. (Fig. 1) But it went to pieces, first because the soil itself was bad, and second, because it had been made worse by improper foundation methods. Yesterday it was boasted here that Mexico has the worst soil in America. I can go it for a





championship of bad soils in Europe and say that Holland and Flanders have perhaps the worst soil in Europe. For the world championship we will compete with the Mexicans.

Since the lock could not be repaired the only thing to do was to put another one adjoining the first one. There was no room to put it somewhere else, than immediately next to the existing lock. But as the latter must remain in service during the time that the new one was being built, engineers of the State Government didn't want to use piles because they feared that the ramming of piles would damage still more the first and make the use of the harbour and dock impossible. So they decided to use compressed air. A caisson was devised by a German firm from Frankfort, Weiss & Freytag acting as consulting engineers for M. Delens, I.C.-A.I.G (Ghent) general contractor.

The caisson is like this. (Fig. 2) The lock is built on four of them, each end on one and two for what we call the sides. The carrying edges are not on the sides but are inside the caisson. There are sharp secondary edges. The result is that the latter cannot carry considerable weight. The reaction is very small on the outside edges, but the reaction on the carrying edges is very important. As a result, the very large girders carrying the whole construction are simply cantilever beams and the moment of flexion is much less than it would be if the carrying edges were on the sides, hence economy of steel.

I think that this is a remarkable principle because it ruins the usual way of thinking, and that is the only way of making progress--to discard the way people have been thinking for centuries and to put something new in its place.

When putting up the caisson, the dead weight before compressed air was put in would make it sink. So they first made a sort of foundation in concrete, with comparatively little cement. When they had such a foundation they mounted the caisson. It could carry the load, and then they destroyed the concrete and the caisson started to sink in. Another feature is that the girders were sticking out of the frame-work, and they later were imbedded in the side-walls of the lock and made everything very rigid.

I think that device has made it possible for the engineers in charge to sleep every night without thinking of what could happen to the lock. It has been a success, and that they won't be obliged to build a third one next to the other.

No. N-17

FOUNDATION CONDITIONS IN MEXICO CITY

Jose A. Cuevas, Consulting Engineer, Paseo de la Reforma, Mexico City

Editorial Note: This paper could not be placed in its proper chronological order due to its late arrival.

General Description of the Basin. Mexico City is located on the southern part of the fill of a basin completely surrounded by mountains. The area of the basin is approximately 7650 square kilometers (about 2950 square miles). In the average its length is 110 kilometers (68 miles approximately) and its width 70 kilometers (about 43.5 miles).

The natural surface of the fill is almost horizontal, at an average height of 2265 meters (7400 feet) above sea level. The two highest peaks of the eastern chain exceed the level of eternal snow which is 4500 meters (14700 feet) on the sea level at these latitudes (about 19° north of the Equator). These peaks are the Iztaccihuatl (the White Dead Woman) the oldest extinct volcano of the region (ending eocene age) and the Popocatepetl (Smoking Mountain), the only still active volcano. Next in importance to the eastern chain of mountains is the southern mountainous region of the Ajusco with its peaks about 4000 meters (13000 feet) above sea level, that is to say 1700 meters (5500 feet) above the city of Mexico.

The summits of the western ridges are lower than the southern. As to the mountains which close the basin towards the north they are the lowest of all and afford easy entrance to the prevailing winds of the region. Within the basin, north of the City, we find a small ridge of hills running westwards to join the central part of the western chain of mountains and eastwards to the middle of the basin where it disappears below the waters of the lake of Texcoco. For the rest, small ridges and isolated hills are seen scattered on the fill of the basin.

The lake of Texcoco and two other still existing ones constitute the remnant of the big lake of Tenochtitlan, the waters of which almost covered the extension of the actual fill of the basin; so that the interior ridge just described was a peninsula and the scattered hills, were prominent islands.

The rock bottom of the basin is very irregular: in some parts it rises on the fill forming hills and in some others it is found hundreds of meters deep.

The Fill of the Basin. The top soil of Mexico City is constituted by a thin layer of vegetable land, by the product of demolitions of old buildings and by the remainder of abandoned masonry foundations: it may be considered as an artificial fill the thickness of which varies from a few centimeters up to a few meters. The volumetric weight of top soil varies from 1400 kg/m^3 up to 2400 kg/m^3 (from 88 to 150 pounds per cubic foot). Below this top soil the material found resembles clay or marl of different rather bright colors that in the course of days turn to a final faded gray or brown color by virtue of the oxidizing action of the air. It is placed in almost horizontal layers separated or divided by thin, more or less pervious strata of sand and clay, which generally occur at a few meters one from another.

The principal characteristics of these colored layers are the following:

1. lightness, which increases with increasing depth, reaching specific mass gravities as low as 1100 kg/m^3 (38#/ft^3);

2. great porosity and void ratio with water contents as high as 3.25 (taking as basis of comparison the weight of solid matter);

3. great compressibility;

4. the capability of the material in its natural state to being shaped and kept with vertical faces as evidenced by the fact that common wells bored to open sky and artesian wells without any lining last long;

5. the low coefficient of permeability of the natural saturated material.

Besides the materials described, others are found which are not so abundant. North of the City quantities of sea or better lake-weeds are found in a place the name of which etimologically reveals the existence of such material ("tiza" is the material and "Tizayuca" the name of the place). Within the City, in its northern part a similar layer, somewhere two meters thick, is found at an average depth of 16 meters. Southeast of the City there is a variety of Turf, resulting from the decomposition of different plants; northwards a layer of about 3 meters thickness in some places is found at about 20 meters depth. Finally we must say that in the marlaceous layers Cypris are found in varying quantities.

It is interesting to make it known that within the basin in Ozumbilla, north of the City, a bore-hole was drilled to a depth of 725 meters (2375 feet) and that after passing the andesitic formation which is lying between 150 and 300 meters depth (490 and 980 feet) soft material was found very similar to the light material which forms most of the City's subsoil.

With a water content as high as 3.21 we cannot think of a cubic or tetraedronic or any other single-grained structure. The material must be of a spongy nature with very large voids. No wonder that theories exclusively based on the single grained structure do not apply to this kind of subsoil. Geological, hydrological and meteorological considerations of the region are important in explaining the nature of this characteristic fill on which the City is resting.

Geological Antecedents.--Hydrological Regime.--Prevailing Winds. The basin was formed by the irregular settling of an andesitic formation which is found in the solid bottom of it in the emerging hills of the valley and all around the basin even in the southern mountains where andesitic formations are covered with basalt lava of recent ages (later pliocene and pleistocene). Some of these basalts are so recent that human remains and pottery have been found below them.

The volcanic nature of the region is the prevailing character of this basin. There are hundreds of extinct volcanoes most of them in the southern mountainous region and many within the fill of the basin. Excepting Popo, we may say that volcanic activity has completely disappeared: it lasted from the end of the eocene age until the latter part of the pliocene: the only remaining manifestations of this past activity may be found in the high temperature of deep artesian waters of different parts of the valley, as high as 67° C. (152° F), but which descends slowly.

All these volcanoes sent into space immense quantities of cinders which, after being deposited, were washed out in the surrounding open valleys, but not so in the closed basin of Mexico City where they accumulated in a very interesting form, by far the most interesting of the fill, both pure and mixed with other fine materials. This peculiar material will be called cineritic fill.

The difference in color which different layers of the fill present when recently cut, indicate the different periods of volcanic activity of these monogenetic volcanoes.

Weathering and erosion explain the presence of ravines on the andesitic formations and of sand, silt and clay deposits near the entrance of ravines in the old lake; only the very light materials suspended in the water entered far into the lake. Melted basalt, erupted at different long intervals by different volcanoes, covered large areas of the mountains of the Ajusco ridge, south of the basin, and cracked at random when cooling. There are no ravines on these basaltic areas, as the rain water disappeared through the cracks.

Prevailing winds blow from north and northeast and beat directly the cool high mountains of the south and southwest. These winds and the geological formations described, make clear some important features of the meteorological and hydrological regimes of the basin of Mexico. The densest forests formed by the tallest trees are found in the valleys southwest of the basin, the most extensive ones cover the skirts of the eastern ridges. Towards the north, the basin is almost free from trees and grass and so it is on large western areas. Streams descending from western ridges are very numerous and generally have the form of torrents making their way down through wild ravines. Erosion and alluvial transportation have been most important on this part of the basin. (At present these waters are being controlled to avoid any flooding danger to the city coming from this source). From the eastern and southwestern valleys we have perennial streams with uniform flow used for hydroelectric purposes.

As we said before, no streams flow on the basaltic formations which almost cover the southern mountains. Rain water which percolates through the cracks runs below the lava, forms subterraneous deposits here and there, feeds pervious layers which project more or less far into the fill. For the most part this water forms springs at different points at the foot of said formations and from there it is pumped for the supply of Mexico City. Streams from the East are less numerous and have less discharge. Pervious alluvial formations penetrating more or less into the almost impervious fill of the basin are saturated by rain water collected on the surrounding mountains of the basin. The hydrostatic head of these artesian waters has descended from a few meters above the ground level down to a few meters below it since 1900 until today.

The Dust Clouds. Another meteorological factor contributing to the formation of the most peculiar part of the fill is what we call "tolvaneras". They are very dense clouds of dust still very common during the last part of the dry season. They are formed on the desert regions in the northern part of the valley previously described and are transported southwards by northerly prevailing stormy winds. When it rains, or else when stormy conditions pass away, the dust settles in tangible layers. These dusty clouds must have been more frequent, lasting and intense in old ages when the whole basin was a tremendously eroded desert.

The Cineritic Fill. If we were dealing only with alluvial material brought into the basin by brooks and torrents we would be unable to explain the almost completed fill of the basin; because alluvial material represents a very small fraction of the volume of the actual fill so that with it alone, the old lake of Tenochtitlan would still exist, hundreds of meters deep, extending throughout the whole area now covered by the City, and even more. That is to say, in such a case the actual City of Mexico, had not yet come into its present existence.

Most of the actual fill, the peculiar jelly-like subsoil of the City, is constituted by very fine vitrious particles, much finer than the particles of clays principally found in the borders of the fill. This cineritic material forms layers more than a hundred meters thick in the deepest parts of the basin. The degree of fineness which this material reaches may be understood if you consider the fact that ashes erupted by a volcano are transported by the wind and found thousands of kilometers from the point of emission. These ashes not only are dispersed in water, but may remain floating in the atmosphere during days, weeks and even months. Many men are still alive who remember the marvellous twilights which during three years following the eruption of the Krakatoe in 1883 were to be contemplated in all continents due to the presence of its fine volcanic ashes in the atmosphere of the whole world.

The rest is very well known by persons familiar with the physical chemistry of the gel formation: honey-comb structures for which we have silts and clays and favorable conditions of sedimentation and flocculent structures (second order honey-comb structures) for which we had fine dust liable to be transported by the waving of waters and by dispersion, plenty of volcanic ashes and fit conditions of flocculation in the increasingly alkalized waters of the lake, are the logical formations of most of the light, extraordinary porous and compressible jelly-like subsoil of Mexico City.

It is interesting to make known here the chemical analysis of the ground water which saturates the subsoil of the Lottery building:

		Parts Per Million
Total solids		520.
Ignited Solids		423.
Total Hardness	as calcium carbonate	155.7
Permanent Hardness	" " "	154.2
Calcium		49.6
Magnesium		10.2
Sodium Potassium		90.5
Chloride		44.5
Sulphate		118.7
Methyl Orange Alkalinity	" " "	162.5
Phenolphthalein "	" " "	7.5
Caustic "	" " "	0.0
Free Carbon Dioxide		0.0
pH Value 7.8		

N.B. 1--Before the time of the drainage operated by the Gran Canal, electrolytes increased every year by the dissolving action of water on surrounding lands, among other causes. Here we have, at the same time, an explanation of how, with a very small amount of solid matter in comparison with the capacity of the basin, most of the fill has rapidly been formed in the course of the most recent geological ages.

N.B. 2--Only long rest was apt to clear up the atmosphere from dust and ashes of eolic transportation which at last fell on the waters of the lake, rain being still necessary to bring into it the finest particles of the caliginous material, including organic matter floating in the air.

Settlements in Mexico City. Since 1880 very careful levellings have been made taking reference points or bench marks on the solid rock formations of the basin: both on the surrounding mountains and on the emergent prominences of the solid rock bottom.

According to the isostatic theory, on account of the continual erosion of the mountains, these are lifting and due to the filling of the basin even the solid rock bottom of it shall be continually sinking; but as far as actual measures warrant, these movements are very slow in comparison with building settlements and may be disregarded for the time being; notwithstanding that the isostatic displacements must have been notably increased by the sinking of the bottom of the basin due to the enormous weights of the cineritic fill and of the water of the old lake and by the upheaval of the volcanic region due to the alleviation of the material erupted (much more than double the weight of the actual cineritic fill of the closed basin).

It is interesting to note that the highest mountain is the old and still alive volcano called Popocatepetl; that next in height are the extinct volcanoes and that the lowest of all are the eroded mountains (west and north of the basin). It is also worthwhile noting that there are clear indications of the sudden uplift of hills the weight of which was alleviated by eruption of internal material and by the "spongification" of the remaining one (due to boiling under water).

Recorded levellings reveal City and building settlements which in some places reach more than a meter (a few feet) from 1880 until the present time. They have made it clear that not only new structures, but also old ones like the Cathedral and the Palacio de Mineria (Mining Palace) are still settling.

An idea of the settlements of the City as a whole may be obtained by considering the changes which the slope of the five oldest sewers of the City and the first 20 kilometers (about 12 miles) of the Gran Canal, have undergone. (See Vol I of Foundation Conference Proceedings, page 295). This general settlement may be visualized in the general apparent uplift of the pipes of deep artesian wells--from 200 up to 250 meters (around 700 feet)--all over the City. This relative lifting of the pipes makes it necessary to cut off the projecting tops of them to avoid troubles in the pumping installations.

FIG: 1

FIG: 2

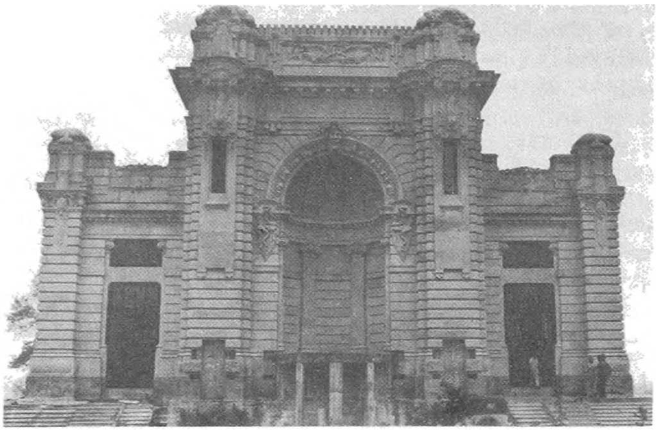


Fig. 3 La Condesa Pumping Plant. The building is set on wooden piles which extend to a firm strata about 21 m deep. It is located near the hills of Tacubaya. Its weight is much less than the weight of earth removed and yet the building has been subjected to slight upward and downward movements with respect to solid rock bench marks. The level of ground water is not constant.

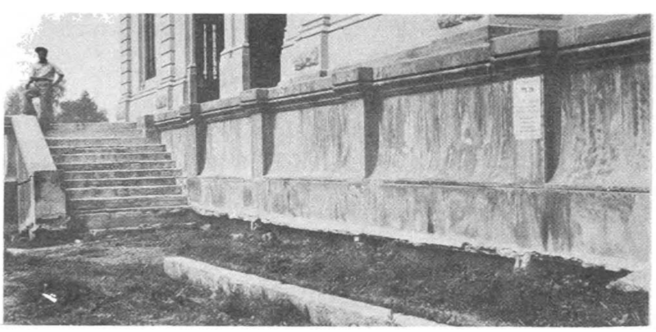


Fig. 4 Settlement of the ground on the east side of La Condesa Pumping Plant.

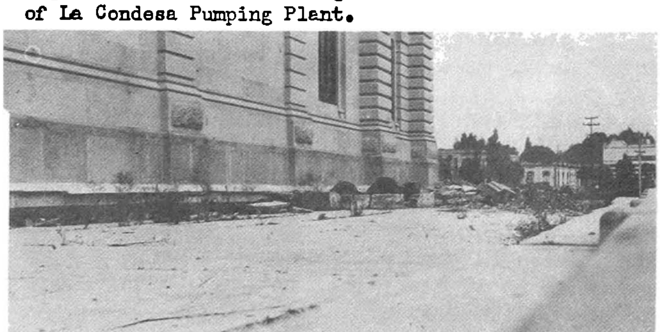


Fig. 5 Settlement of the ground on the north side of La Condesa Pumping Plant.

The drainage of the soil through the more or less pervious strata which was brought about when the new sewerage system (1900) was built and the pumping of artesian water, actually has aggravated the general settlement of the City. Earthquakes, particularly those of trepidatory character, have increased the settlements, specially on the softer and deepest parts of the fill.

A general contraction of this fill, has been accepted by our geologists as an explanation of the fact that a geographic base line of 7500 meters (about 8200 yards) was found shorter when it was measured again, years later. This contraction is ascribed to the drainage of the basin through the Gran Canal which practically converted it into an open valley since 1900 and also to the active surface evaporation which takes place in this valley of so high altitude and of so small latitude.

The settlement of a construction is not limited to the period of building, but increases continually at an indefinitely decreasing rate: the time-settlement graph seems to be asymptotic to the time axis in most cases.

When the height of a building is uniform, especially when it is poorly designed or when horizontal dimensions are large enough, it generally settles more at the center--as it is well known--and it frequently happens that cracks appear as a consequence of the sagging.

It is worth noting the fact that in some old buildings the sagging of the lower stories is more noticeable than the sagging of the higher. The explanation is that the settlement actually began with the erection of the building and that without correcting it, the succeeding stones were cut to measure so as to reduce gradually the sagging on the higher layers until they had them transitory horizontal. This is the reason why in Mexico City we may find the stones strained to the limit by intervening secondary stresses; but that actual deformations not always reveal the total differential settlements undergone.

The settling of a building affects the surrounding subsoil and when different buildings are separated by open spaces two normal cases may occur as seen in Fig. 1 and 2. Exceptions to the general rule are found due to local circumstances. One very notable exception is constituted by the pumping plant of "La Condesa", which instead of settling has apparently lifted up more than once, and around which the ground actually settles as both the feeding and the exhaust pipes notwithstanding that its weight--including water contained--is lighter than the earth excavated and removed far from the existing trench (see Fig. 3, 4 and 5). Now this case, is still an exception; but not a misleading one as every aspect of the phenomenon may be clearly explained in the light of recent experiments and studies.

The settlements of the Y.M.C.A. building and other buildings of the same block have been aggravated long ago by pumping large quantities of water from an artesian strata 210 meters (about 700 feet) deep. Recently it was necessary to lower the level of ground water from its normal depth of 1.80 meters (6 feet) down to a depth of 5.40 meters (18 feet) by pumping. This pumping influenced the settlements more than the deep one. The heavy mass concrete block poured on the top of the 8" pipe of the artesian well and rigidly connected with the building prevented the pipe from lifting and due to the settlement of the building it was bent. Motor and pump vibrations destroyed the structure of the cineritic fill, forming a hole three feet in diameter all along the pipe and concentric to it. This fact proves the capacity of this cineritic material not only to stand steep, but to sustain itself with negative inclinations, notwithstanding being 100% saturated under certain circumstances.

Upheaval of Ground. When a building is taken down and the debris is removed, the ground always lifts up: the effects on neighbouring buildings present two normal forms closely related with the two cases illustrated before (see Fig. 6 and 7). In the case of Fig. 6 the usual shoring of adjoining buildings is not only useless, but of negative effect. The uplift-time graph has the same asymptotic character described before for the reverse case.

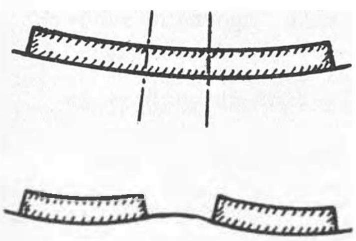


FIG: 6

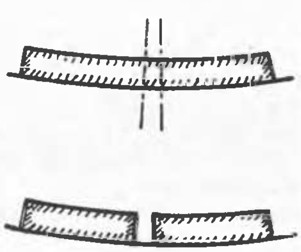


FIG: 7

When earth is excavated and removed from an excavation, the upheaval of the remaining soil also presents itself. If the excavation is sufficiently ample and deep, the bottom and the borders lift in an apparent form, although the lifting of these is diminished and even reversed by the shrinkage following the drainage of surrounding soil and by the compressive action of neighbouring loads.

It is important to note that cuts may be very steep when the remaining material is not disturbed: we are dealing with a cohesive material of very low "modulus of elasticity". As in preceding cases the unloading of the bottom causes an immediate upheaval of it, and in the course of time it increases in an asymptotic form. All these aspects of the behaviour of Mexico City subsoil have been made clear, demonstrated and measured by means of "submarks" called "testigos".

Sometimes it happens, as it actually happened when the excavation of the Pumping Plant of La Condesa was executed and when the digging of the Gran Canal was done, that the bottom of the excavation suddenly bursts. To avoid such an accident, one or a number of the following precautions have been successfully taken to avoid long delays, undue expenses and also uncertain and disappointing conditions:

1. to remove the excavated material far away from the excavation;
2. to lower the hydrostatic pressure in the more or less pervious layers below the impervious bottom of the excavation by means of drilled bores lined with perforated pipes, reaching layers sufficiently deep;
3. to continue the excavation by dredging, avoiding pumping and keeping the height of ground water practically at its normal level;
4. to keep the excavation at a convenient pace, in accordance with the indications of an up-to-date uplift-time graph, the slope of which is carefully observed.

If these uplift movements are disregarded it is no wonder that subsequent settlements occur, notwithstanding the load placed on the bottom of the excavation may be lighter than the earth excavated and removed.

It has been a privilege for the inhabitants of Mexico City to bring their concerns on foundation problems before this audience for consideration. Suggestions to help us in completing an exhaustive program of study will be greatly appreciated. It has been an honor to me, as a messenger of Mexican civil engineers, to interpret before you the behavior of the subsoil of Mexico City.