island first appeared near the middle of the canal and later formed a complete barrier, 250 ft wide at the water surface, rising 65 ft above the water level. The canal remained closed to traffic for 7 months until April 15, 1916.

The Cucaracha slide again became active on August 30, 1916, and closed the canal to traffic for 8 days. The East Culebra slide started moving again in January, 1917, and closed the canal to traffic for 2 days. No further closures of the canal were experienced until March 20, 1920, when the Cucaracha slide again broke loose to close the canal for 4 days. In 1931 the canal was closed to traffic for 2 days while a small slide in the East Culebra area was removed. This was the last time that the canal was closed because of slides.

Design of Slopes

Cucaracha Formation.—All the worst slides of the original canal construction involved the weak, slickensided clay shale of the Cucaracha formation. Since the alinement for a new sea-level canal is through an area where the Cucaracha formation is present, the design of slopes in this material is of the greatest concern.

The Cucaracha slope-design problem was studied intensively during the Third Locks investigations, since an important part of the excavation for the new Pedro Miguel locks would have been in this material. The studies included laboratory and field tests of the undisturbed clay shales, analyses of the slides along the existing canal, and the analysis of an existing stable slope in the Cucaracha formation.

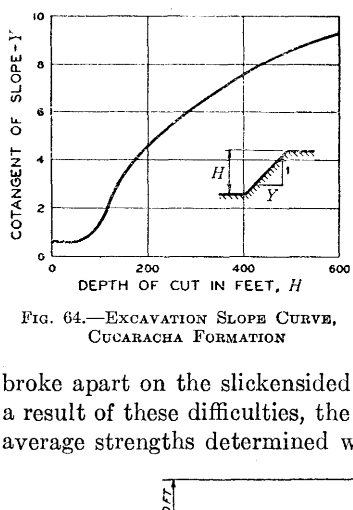


Fig. 64.—Excavation Slope Curve, Cucaracha Formation

Difficulties were encountered in obtaining suitable laboratory samples from the initial core-drilling operations, as the cores recovered from the weaker horizons were often badly broken. The subsequent use of a heavy mud slurry in place of drill water reduced caving and increased the percentage of recovery of undisturbed samples through the more critical and weak horizons.

Many of the samples that were recovered broke apart on the slickensided surfaces when being prepared for testing. As a result of these difficulties, the weakest materials could not be tested, and the average strengths determined were regarded as high.

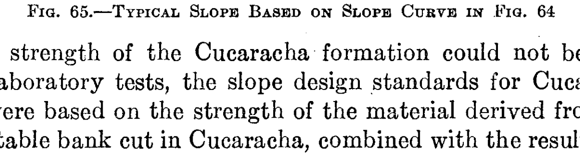


Fig. 65.—Typical Slope Based on Slope Curve in Fig. 64

Since the strength of the Cucaracha formation could not be determined reliably by laboratory tests, the slope design standards for Cucaracha (Figs. 64 and 65) were based on the strength of the material derived from analytical studies of a stable bank cut in Cucaracha, combined with the results of a simple laboratory test to measure the friction that might be expected along the slickensides in the material.

The tests to determine the friction were made in a standard direct-shear machine on specimens cut from solid cores of sound clay shale, polished to simulate slickensides, and brought into contact under water. The minimum angle of friction determined by three series of such tests on three different samples, under various conditions of normal load, was 10° which compares with a value of 13.5° obtained for the angle of internal friction from laboratory compression tests.

The stable bank referred to previously is a cut about 200 ft deep in the Cucaracha formation on the west bank of the canal, just south of Zion Hill, which has an average slope of 1 vertical on 2.7 horizontal. This bank has never been disturbed by slides but is composed of material similar to the Cucaracha material that would be encountered in the excavation of a sea-level canal. Computations were made to establish the cohesion needed for stability of the bank along the most critical sliding arc, assuming a factor of safety of 1.0 and a friction angle of 10°, as found from the laboratory tests. In this way a value for the cohesion of the formation as a whole was obtained which provides a reasonably sound basis for analysis of other cuts in similar material, since the effects of weak and strong bedded units, slickensides, joints, fractures, and gouge areas are automatically accounted for.

In the analyses, the ground-water table was considered to have been at the level of a bed of material known to provide good drainage, and therefore the effect of sudden drawdown, applied as a simple approximation of the forces of steady seepage, was limited to the mass below this bed. The analyses for a factor of safety of unity resulted in a value of 16 lb per sq in. (2,300 lb per sq ft) for the cohesion of the Cucaracha formation. This value was checked by an analysis of the first known shear slide that involved the Cucaracha formation—that of October, 1907—when Gaillard Cut was about 100 ft deep and presumably was being excavated to a slope of 3 vertical on 2 horizontal. The value has also been checked by analytical studies of the East Culebra and West Culebra slides.

The adopted strength values, cohesion equal to 16 lb per sq in., friction angle equal to 10°, and a factor of safety of 1.3, were used in developing the slope curve shown in Fig. 64. Sudden drawdown over the entire height of cut was assumed for the Cucaracha slopes. This assumption simplified the analyses and yet was somewhat conservative with regard to steady seepage out of the bank, a condition which might prevail after excavation. The part of this curve for cuts deeper than 200 ft was developed from data published by D. W. Taylor.[45] The part of the curve for cuts less than 200 ft deep is empirical and is based on recommendations made by Mr. MacDonald on the basis of his broad experience with the Cucaracha formation. Slopes for cuts less than 200 ft deep can justifiably be flatter than theoretically necessary, because the weakening effect of from 20 ft to 40 ft of weathered rock and residual clay overburden may be of some importance in shallower cuts.

[45] "Stability of Earth Slopes," by D. W. Taylor, *Journal*, Boston Soc. of Civ. Engrs., July, 1937, p. 197.

Atlantic Muck.—The extreme instability of the Atlantic muck was first observed in the early 1900's during the excavation for the north approach walls of the existing Gatun Locks. The muck was excavated to El. −55 on slopes of 1 vertical on 5 horizontal; but, before the dredging was completed, the material had slid in some places making the slopes as flat as 1 vertical on 13 horizontal. After the excavation was dewatered, when the pile foundation for the approach wall was about half completed, the east bank gave way and covered the greater part of the foun ation with mud to a depth of from 6 ft to 18 ft. The final slope was 1 vertical on 20 horizontal.

In the design of the Third Locks, a similar slope problem was encountered in the excavation of an extensive muck area for the construction of the approach walls of the new Gatun Locks. Unconfined and triaxial compression tests, supplemented by direct-shear tests, were made on representative undisturbed samples of the material. The first tests on the muck were run quickly; the entire time of the test, after consolidation of the sample to overburden pressure had been completed, was only from 10 min to 20 min. It was later found that lower results were obtained if the tests were run at a slower loading rate. A critical loading rate of 1.6 lb per sq in. per 15 min was found to give the lowest strengths and was adopted for all subsequent tests. The duration of tests for that loading rate varied from 60 min to 300 min. The results of these tests showed conclusively that, independently of the time of loading, the strength of the muck depends on the degree of consolidation of the material at the time of shearing. Thus, within any muck deposit, the strength of the muck is greater at increasing depths, since the consolidating pressures increase with depth. Fig. 66 shows data from the final series of tests, made at the critical

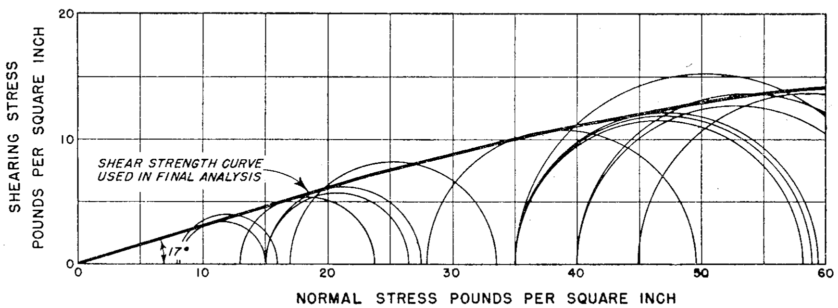


FIG. 66.—SHEAR STRENGTH CURVE FOR ATLANTIC MUCK

loading rate, which indicate the strength characteristics of the muck. (Each circle in Fig. 66 represents one triaxial compression test on a sample consolidated under pressures equal to or greater than the overburden loads and then loaded at the rate of 1.6 lb per sq in. per 15 min to failure.) In the Third Locks design studies for slopes in Atlantic muck to be cut by dredging and left permanently submerged, it was found that stability was influenced more by the height of the bank above permanent water level than by the depth of cut below water level—because the saturated weight of the muck above water level is about 90 lb per cu ft, whereas the weight of the submerged material is only about 28 lb per cu ft. The proper slope for a permanently submerged cut in muck, therefore, is dictated largely by the height of the top of the cut above water. The slope adopted for use in the Gatun Locks area, where the top of muck was less than El. 10 and the total depth of cut was greater than 30 ft, was 1 vertical on 2.5 horizontal. Such slopes were actually cut during the period from 1941 to 1945 and have remained stable since that time.

A large part of the sea-level canal between the Atlantic entrance and Barro Colorado Island would require cuts in Atlantic muck which would be made by dredges operating on the present level of Gatun Lake. After lowering Gatun Lake to sea level, the top of the muck banks would be exposed above water level, in some areas as high as El. 35. Undisturbed samples of the muck underlying Gatun Lake were tested and found to have strength characteristics approximating those shown in Fig. 66.

One cross section for sea-level canal excavation through muck in the Gatun Lake area, Fig. 67, was designed to meet all conditions. This slope was ana-

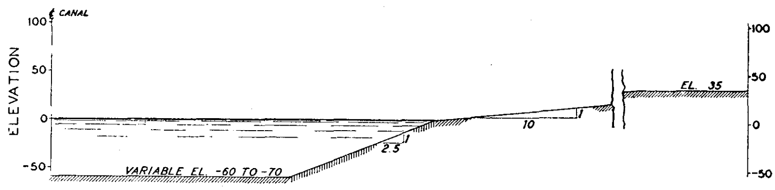


FIG. 67.—EXCAVATION SLOPES IN MUCK

lyzed by the Swedish circular arc method, using the shear strength curve of Fig. 66, and the minimum factor of safety obtained was 1.3. The slope of 1 vertical on 2.5 horizontal below water level results from the desire to maintain as uniform a channel cross section as possible, in keeping with the cross section through the rock-cut areas, and from the successful experience with this particular slope in the Gatun north approach channel. The uniform slope of 1 vertical on 10 horizontal above water level would simplify the dredging and is amply safe for top-of-slope elevations as high as El. 35.

Other Rocks.—Slope-design standards for materials other than the Cucaracha and Atlantic muck formations were developed empirically. These can be classified into three groups: (1) Soft rocks—the Culebra, La Boca, and Las Cascadas formations; (2) medium rocks—the Gatun, Bohio, and Caimito formations; and (3) hard rocks—basalt, the Pedro Miguel agglomerate, and Bas Obispo formations. Slides in these materials, where encountered in the existing canal, were uncommon and usually resulted from structural causes.

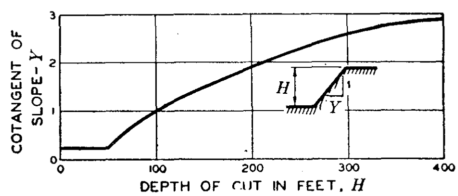


FIG. 68.—EXCAVATION SLOPE CURVE, CULEBRA FORMATION

The slope curve for the first and weakest of the three groups is shown in Fig. 68. This curve is based on recommendations made by Mr. MacDonald and on experience with previous excavation in the Culebra formation. A typical slope based on the curve in Fig. 68 is shown in Fig. 69.

Fig. 70 shows the standards for excavation slopes in the other two groups of materials, medium and hard rocks. The two standards differ only in the vertical interval between berms. The channel slope is 3 vertical on 2 horizontal up to El. 15. Above a 45-ft berm at this elevation the slopes are 12 vertical on

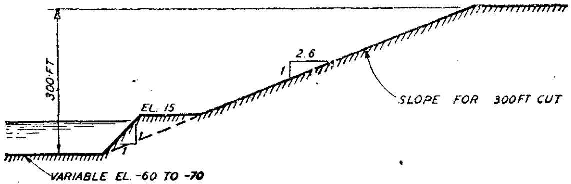
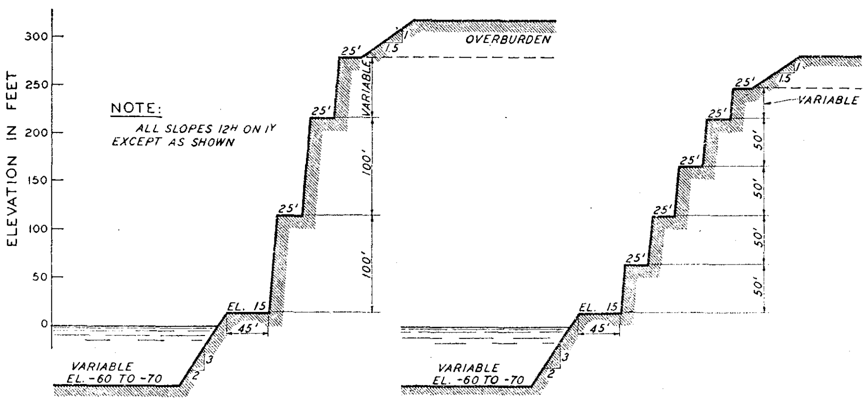


FIG. 69.—TYPICAL SLOPE BASED ON SLOPE CURVE IN FIG. 68

1 horizontal with 25-ft berms every 50 ft in elevation for the medium rocks, and every 100 ft for the hard rocks.

Considering stability only, it is possible that, in any of the medium or hard rocks, vertical cuts as high as 500 ft would stand without danger of major failures. However, rockfalls are always a possibility and would be particularly dangerous during construction. A 45-ft berm at El. 15 is provided to prevent



(*a*) BASALT, AGGLOMERATE, AND BAS OBISPO (HARD ROCKS)

(*b*) GATUN, BOHIO, AND CAIMITO (MEDIUM ROCKS)

FIG. 70.—EXCAVATION SLOPES FOR MEDIUM AND HARD ROCKS

falling rock from reaching the channel. The 25-ft berms above El. 15 would provide a reasonable degree of safety during construction.

Channel Slopes.—The channel cross section should be as nearly uniform as possible to provide the best navigation conditions. Rock slopes for a sea-level canal designed only for stability would vary from 12 vertical on 1 horizontal for hard rocks to as flat as 1 vertical on 9.4 horizontal for the Cucaracha formation. To reduce this extreme variation for the benefit of navigation, the slopes below El. 15 were modified as follows: In hard rocks, the channel slopes would be 3 vertical on 2 horizontal; in Cucaracha and other soft rocks, a channel slope of 1 vertical on 1 horizontal would be used as indicated in Fig. 65 and 69.

Except for this modification, the over-all slope in these soft materials would be designed by the slope curves.

Dynamic Loads

Large Explosions.—The effects of large bomb explosions in the vicinity of excavations were not taken into account in the development of the slope-design standards described previously. Investigations were made, however, of the effect that large explosions in air and on or under the ground might have upon the stability of slopes. Research was conducted by Harvard University to investigate the effects of explosive forces on the strengths of soils and rocks. This research is reported in the eighth Symposuim paper.

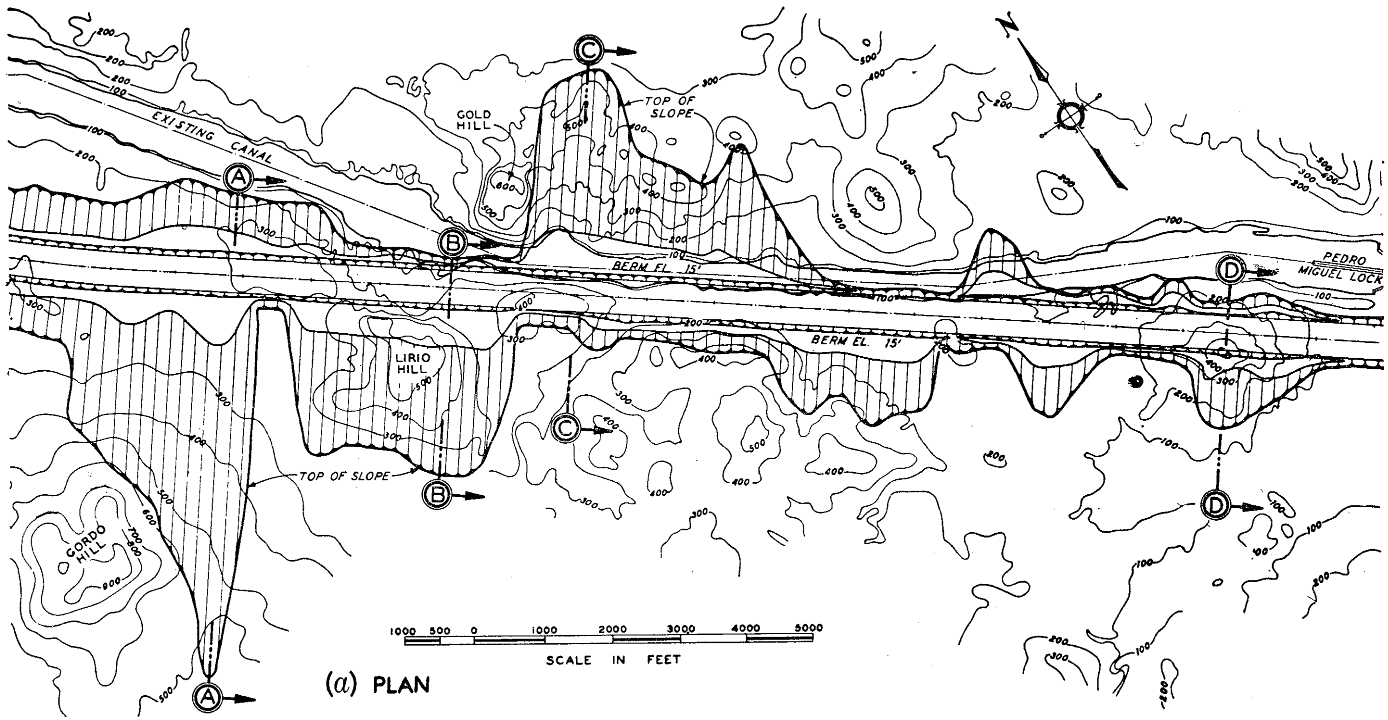
The studies indicate that sliding failures caused by dynamic loads on a statically safe slope would probably not result in closure of the canal. From present evidence, flattening the slopes beyond the requirements for static stability, therefore, is not believed to be necessary.

Earthquake Hazards.—According to information furnished by L. Don Leet, Professor of Geology at Harvard University, landslides do not occur at distances greater than 50 miles from the epicenter of the largest earthquakes known to have occurred, or at more than 20 miles from one as large as that in Japan in 1923. The computed accelerations which produced slides range from around 0.5 g to 0.7 g. The earthquake history of the Isthmian region around the Canal Zone indicates that no serious danger exists of an earthquake large enough and near enough to place the Canal Zone within such a landslide radius. The closest major active zone on record has been off the Los Santos peninsula, more than 100 miles from Balboa. Absolute prediction, either positive or negative, of the time and place of occurrence of earthquakes cannot be made. It is concluded from the study, however, that earthquake-induced landslides in cuts for the Panama Canal would be improbable; therefore, no allowance for earthquake forces has been made in the design of canal slopes.

Application of Slope-Design Standards

The design standards described previously for excavation slopes under conditions of static loading are sufficiently detailed for quantity estimating and construction planning purposes. The application of these standards to the 4-mile section of a sea-level canal through the Continental Divide north of Pedro Miguel is illustrated in Fig. 71. Approximately 30% of the total required excavation is concentrated in this area. This section of the canal would also include the deepest cuts.

In the development of the standards illustrated by the slopes shown in Fig. 71, minimum strength values have been used for each of the geologic formations encountered. Before actual construction of a sea-level canal, more detailed geologic data would be available, and the slopes would be modified to take advantage of stronger phases of some formations. It is believed that any modification would result in a reduction of excavation quantities.


EXISTING CANAL
GOLD HILL
TOP OF SLOPE
BERM EL. 15'
PEDRO MIGUEL LOCK
LIRIO HILL
GORDO HILL
TOP OF SLOPE
BERM EL. 15'
N
1000 500 0 1000 2000 3000 4000 5000
SCALE IN FEET

(a) PLAN

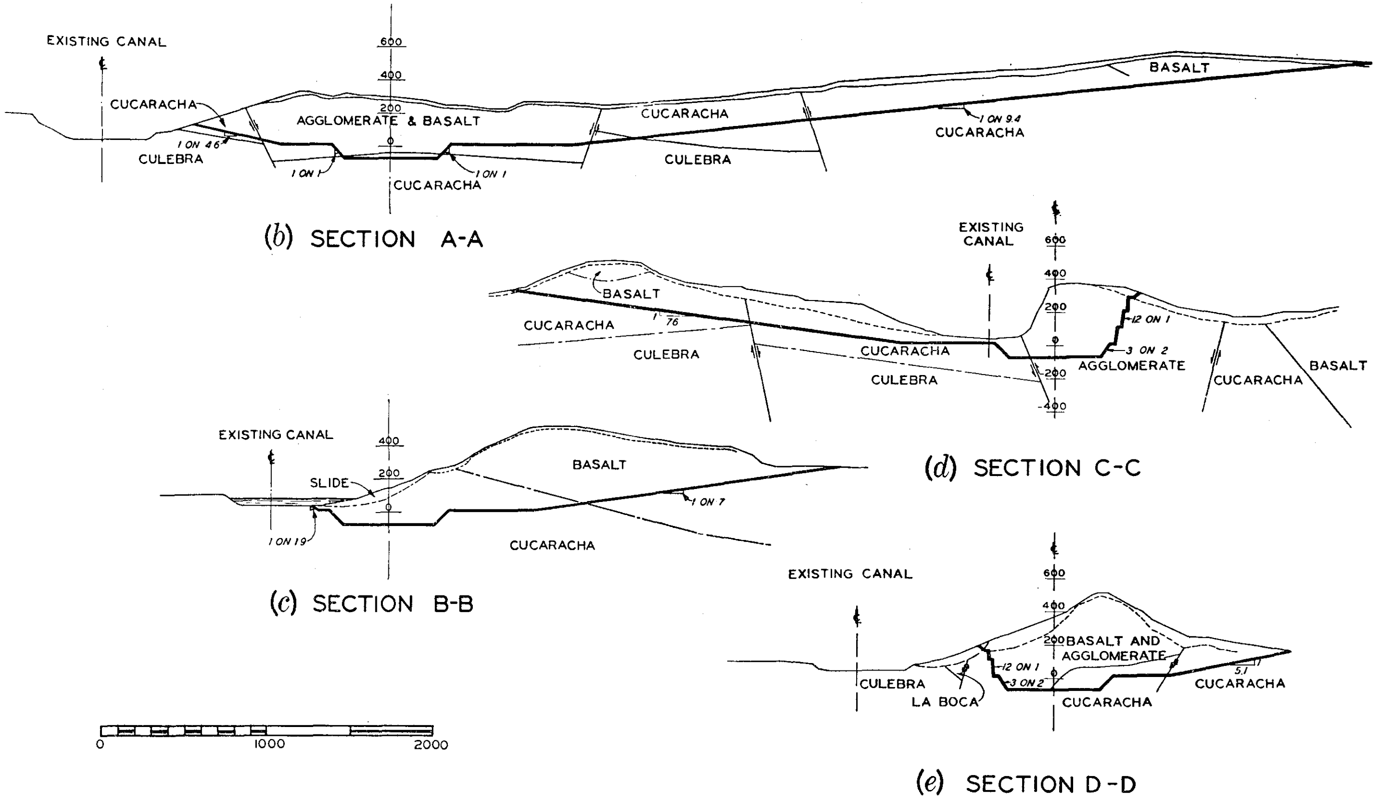


FIG. 71.—EXCAVATION LIMITS IN DEEP CUT SECTION

Conclusions

Although large slides occurred during and following construction of the present Panama Canal, major slides would be prevented in the proposed Panama sea-level canal by excavating the channel banks initially to the proper slopes.

The analytical methods and the testing employed for the design of safe slopes conform with established methods that had not been developed when the present canal was constructed. Studies of the slides that occurred during canal construction were of value in the design of stable slopes for the Cucaracha formation.

The design standards presented in this paper are considered to produce slopes that would be stable under static loading. Any slides that might be initiated by dynamic forces from earthquakes or large explosions would not be expected to cause closure of the canal.

STRENGTH OF SOILS UNDER DYNAMIC LOADS

By A. Casagrande,[46] M. ASCE, and W. L. Shannon,[47] Assoc. M. ASCE

Synopsis

This paper describes apparatus developed and results of tests performed to investigate the strength characteristics of soils and soft rocks under dynamic loads.

Introduction

In connection with studies of the stability of slopes under the effects of bombing, laboratory investigations on the strength of soils and soft rocks under dynamic loading are being conducted at Harvard University for The Panama Canal.

This investigation is expected also to benefit other engineering problems in which soil is subjected to dynamic loading, such as the effects of earthquakes on dams and their foundations, or the effects of transient loading by fast moving traffic on airfield and highway pavements and the underlying materials.

Conventional strength tests on soils are either unconfined compression, or triaxial compression, or direct-shear tests. Any of these tests can be performed with either a controlled stress or a controlled strain loading apparatus. Loading of the specimen in such tests is performed over a period of at least several minutes. Such tests will be referred to herein as static strength tests, to distinguish them from the dynamic tests described in this paper.

It has been recognized that the strength of soil increases as the rate of loading increases. For example, in connection with the design of the third locks for The Panama Canal, a series of triaxial compression tests was conducted to determine the strength of undisturbed, soft organic clay by producing failure within a range of from 1.7 min to more than 7 hours. These tests indicated that the strength at the fastest rate of loading was about 40% greater than that at the slowest rate. D. W. Taylor,[48] Assoc. M. ASCE, investigated the strength of a clay that was remolded at the liquid limit and then consolidated under 4.22 kg per sq cm. Failure was produced within the range of from 4 min to 8 days. In these tests the strength of specimens that were loaded to failure quickly was found to be about 25% greater than the strength of specimens that were loaded slowly.

Investigations have been performed on metals to determine their strength at various rates of strain. One comprehensive series of tension tests was performed by M. J. Manjoine[49] on a mild steel within the range of from 1×10^{-6}

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[46] Prof. of Soil Mechanics and Foundation Eng., Graduate School of Eng., Harvard Univ., Cambridge, Mass.

[47] Research Associate in Soil Mechanics, Graduate School of Eng., Harvard Univ., Cambridge, Mass.

[48] "Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils," U. S. Waterways Experiment Station, Vicksburg, Miss., April, 1947, p. 95.

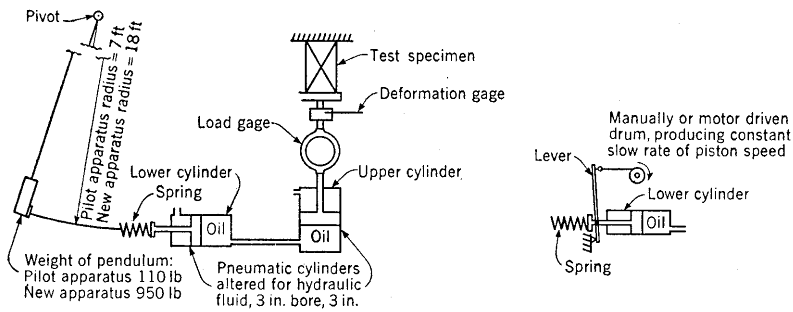
[49] "Influence of Rate of Strain and Temperature on Yield Stresses of Mild Steel," by M. J. Manjoine *Journal of Applied Mechanics*, December, 1944, p. A-211.

strain per sec to $1 \times 10^{+3}$ strain per sec, which corresponds to a range of time necessary to reach the ultimate strength of approximately 2.3 days to 0.0002 sec. The ultimate strength of specimens tested in the shortest time was found to be about 50% greater than that of specimens tested in the slowest time.

For fast transient tests on soils and soft rocks, it was necessary to develop apparatus for applying dynamic loads and for measuring and recording the loads applied and the resulting deformations of test specimens.

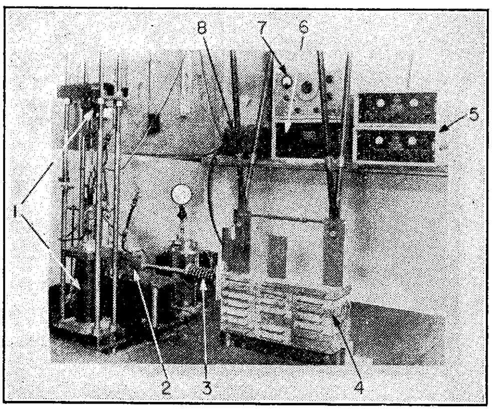
Apparatus for Applying Transient Loads

After a comprehensive review of dynamic testing apparatus developed for various purposes, the writers realized that none of these apparatus would be suited for this investigation, and they were obliged to develop new apparatus



(*a*) PENDULUM APPARATUS (*b*) MODIFICATION TO PENDULUM

Fig. 72.—Pendulum Loading Apparatus



LEGEND

1. Loading frame (see Fig. 74)
2. Lower cylinder
3. Spring
4. Pendulum, weighted to 700 lb
5. Strain indicators
6. Power supply
7. Oscillator
8. Oscillograph

Fig. 73.—General View of Pendulum Loading Apparatus, with Recording Instruments in the Background

for applying transient loads. The type of loading desired was a transient load in which the test specimen is subjected to a rapid loading and unloading, simulating the effect of the first stress wave created by an explosion. As a criterion for the speed of load application, it was found convenient to define the time of loading as the difference in time between the beginning of test and the time at which the maximum compressive stress is reached. The value for the fastest time of loading for use in this investigation was determined in consultation with H. M. Westergaard, M. ASCE, and L. Don Leet, special consultants to The Panama Canal. The value thus decided on was 1/100 sec. The time for the slowest loading was determined by the desire to overlap with the fastest loading time used in static strength tests.

Three different types of apparatus for applying transient loads in triaxial compression and unconfined compression tests were developed simultaneously,

LEGEND

1. Upper hydraulic cylinder and piston
2. Shielded cable to deformation gage
3. Deformation gage
4. Tilting cap
5. Reaction for deformation gage
6. Unconfined compression specimen
7. Shielded cable to load gage
8. Load gage
9. Tie rods

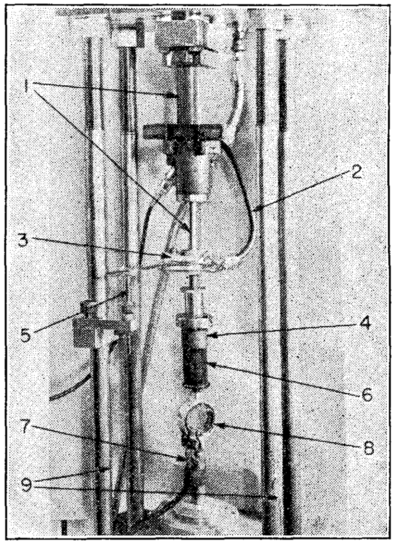


Fig. 74.—Close-Up of Loading Frame of the Pendulum Apparatus, Showing the Arrangement for the Unconfined Transient Compression Test

since it was not certain what method of load application would be best suited to this investigation. It finally was found that all three types were needed because they supplemented one another in the range of time of loading for which each type was best suited.

Pendulum Loading Apparatus.—Figs. 72 and 73 show a diagram and a photograph of the pendulum loading apparatus which utilizes the energy of a pendulum that is released from a selected height and strikes a spring connected to the piston rod of a 3-in. bore cylinder. This lower cylinder, in turn, is connected hydraulically to an upper cylinder of the same bore, which is mounted within a loading frame. Fig. 74 is a detailed view of the loading frame of the pendulum apparatus.

The time of loading for which this apparatus was found best suited ranges between 0.01 sec and 0.05 sec.

The Falling Beam Loading Apparatus.—The loading apparatus in Fig. 75 utilizes the unconfined compression test apparatus of the "universal soil loading machine"[50] for the application of a transient load. The apparatus consists essentially of a beam with a weight and rider, a dashpot to control the velocity of the fall of the beam, and a yoke for transmitting the load from the beam to the specimen. A small beam mounted above the yoke counterbalances the weight of the beam. This apparatus was found to be suited for a relatively long time of loading, ranging from 0.5 sec to about 300 sec.

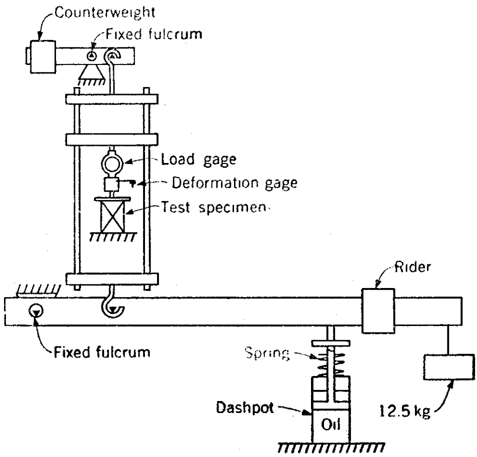


Fig. 75.—Falling Beam Loading Apparatus

The Hydraulic Loading Apparatus.—The loading apparatus in Figs. 76 and 77 consists of a constant volume vane-type hydraulic pump connected to a hydraulic cylinder through valves by which either the pressure in the cylinder or the volume of liquid delivered to the cylinder can be controlled. The peak load that can be produced with this

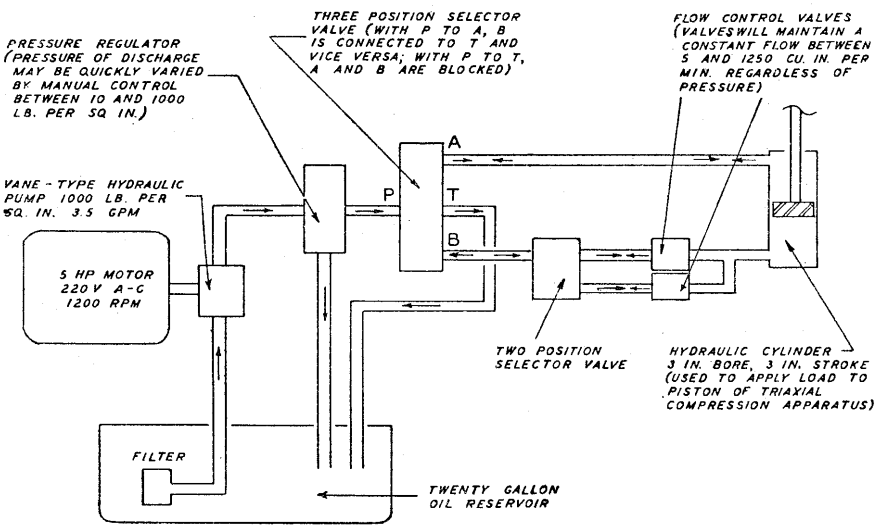
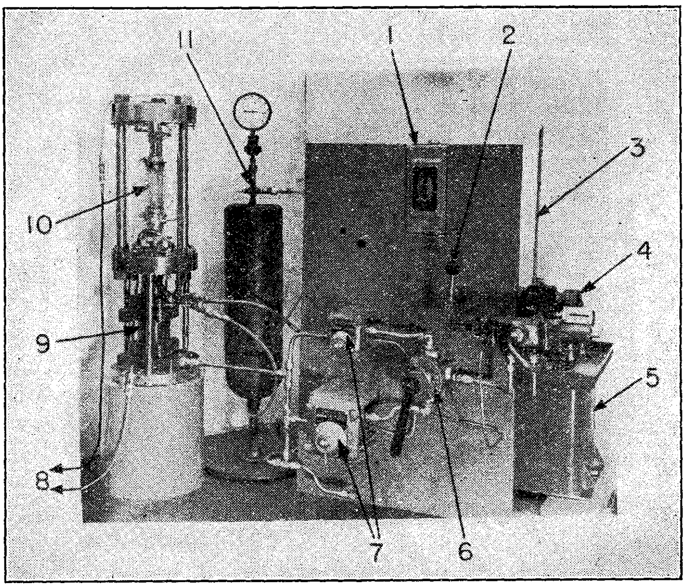


Fig. 76.—Diagram of Hydraulic Loading Apparatus

apparatus is much greater than can be obtained by either the falling beam or the pendulum types of loading apparatus.

[50] "Recent Developments in Soil Testing Apparatus," by P. C. Rutledge, in "Contributions to Soil Mechanics 1925-1940," Boston Soc. of Civ. Engrs., Boston, Mass., 1940, pp. 243-250.

This apparatus is used for testing soft rocks from the Canal Zone with a time of loading between 0.05 sec and any desired slow loading.



LEGEND

Point	Description	Point	Description
1	Starter switch for 5-hp electric motor which is behind panel	7	Flow control valves
2	Three-position selector valve	8	Cables from load and deformation gages (inside pressure chamber) leading to recording instruments
3	Handle of pressure regulator	9	Hydraulic cylinder
4	Vane-type hydraulic pump operated by electric motor	10	Test specimen inside transparent pressure chamber
5	Oil reservoir		
6	Two-position selector valve	11	Air pressure reservoir

FIG. 77.—PHOTOGRAPH OF HYDRAULIC LOADING APPARATUS

APPARATUS FOR APPLYING STATIC LOADS

Two types of loading apparatus were used for determining the static compressive strength.

The Fairbanks Scale Loading Apparatus.—This type consists of a conventional 500-lb Fairbanks platform scale equipped with a loading yoke and a mechanical jack.[51] It is used extensively in performing soil tests with stress control.

[51] "The Soil Mechanics Laboratory at Harvard University," by P. C. Rutledge *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., June, 1936, Vol. II, pp. 85–97, particularly Figs. 14 and 15.

The Hydrostatic Loading Apparatus.—This type consists essentially of the two hydraulic cylinders of the pendulum apparatus and a motor-driven drum and lever with which the travel of both hydraulic pistons can be controlled at a constant, slow rate.

Triaxial and Unconfined Compression Apparatus

All triaxial compression apparatus used in this investigation for transient load tests have been adapted from triaxial apparatus previously constructed at Harvard University for static testing.[52] One of these is a vacuum type and three are compression types. The vacuum type of apparatus is limited to

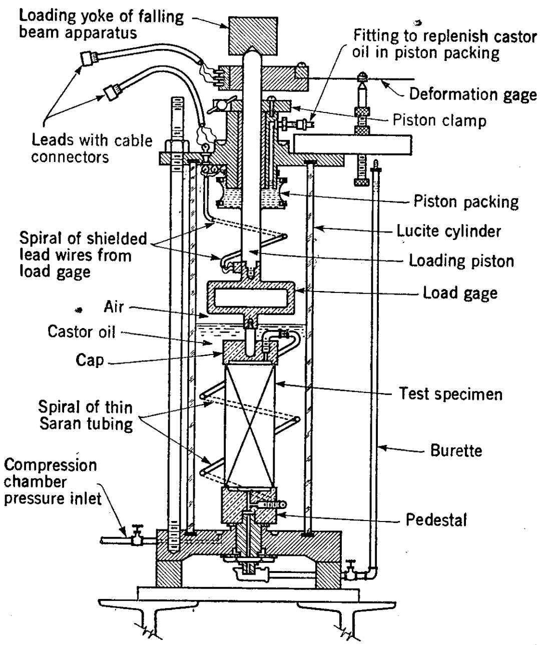


Fig. 78.—Diagram of Triaxial Transient Compression Apparatus

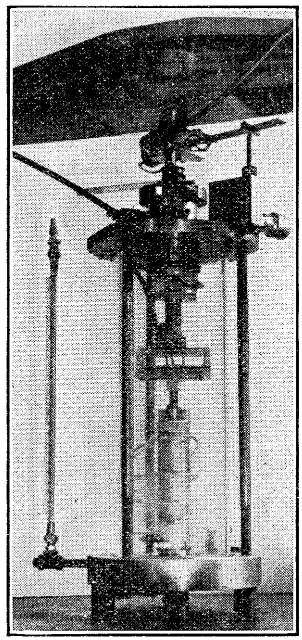


Fig. 79.—Photograph of Triaxial Compression Apparatus

tests on dry, noncohesive soils under a minor principal stress less than one atmosphere. The compression-type apparatus illustrated in Figs. 78 and 79 is suitable for tests on either cohesive or noncohesive soils under a minor principal stress up to about 6 kg per sq cm.

Figs. 72, 73, and 74 show the pendulum apparatus assembled for unconfined compression tests; and Fig. 75 shows the falling beam apparatus similarly assembled.

Apparatus for Measuring and Recording Transient Loads and Deformations

A comprehensive study was made of instruments available for measuring and recording transient compressive forces and deformations. In addition,

[52] "Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils," U. S. Waterways Experiment Station, Vicksburg, Miss., April, 1947, pp. 130–154.

several months of intensive work were required to develop suitable load and deformation gages. To measure and record rapidly changing loads and deformations, metal-electric (SR-4) strain gages with companion strain indicators and oscillographs were found most suitable, in part because such equipment was readily available. A brief description of this instrumentation follows:

Load Gage.—For measuring load, a load gage of rectangular or cylindrical shape is used, with four metal-electric (SR-4) strain gages mounted on the inside face. (The irregular mass on the interior surfaces of the load gage in Fig. 80 is wax placed there for the protection of the SR-4 gages.) Load gages are also shown in Figs. 72, 74, 75, 78, and 79.

Deformation Gage.—For measuring deformation, a thin flexible steel spring cantilever is used (Fig. 81) with metal-electric (SR-4) strain gages mounted on

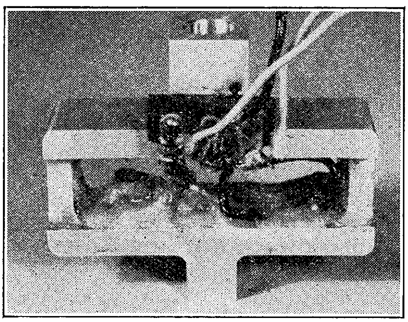


FIG. 80.—LOAD GAGE

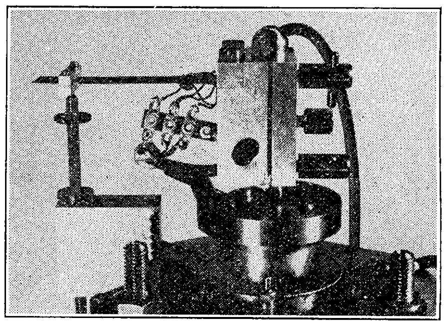


FIG. 81.—DEFORMATION GAGE

the cantilever, the base of which is clamped to the loading piston. The tip of the cantilever reacts against an adjustable screw which is mounted on the head plate of the triaxial compression apparatus. Deformation gages are also shown in Figs. 72, 74, 75, 78, and 79.

Recording Apparatus.—Equipment for amplifying and recording the signal produced by the SR-4 strain gages consists of two strain indicators,[53] an oscillator, a power supply, and oscillographs. The strain indicators contain two arms of a Wheatstone bridge and a carrier type amplifier. The other two arms of the Wheatstone bridge are the SR-4 strain gages on the load or deformation gages. This equipment can be seen in the background of Fig. 73.

MATERIALS TESTED AND TECHNIQUE OF TESTING

Static and transient compression tests have been performed to date on the following materials:

(*1*) *Manchester* (*N. H.*) *Sand.*—This is a clean, medium sand obtained by screening from a glacial-fluvial deposit and contains only the fraction between 0.42 mm and 0.21 mm. It consists principally of subangular quartz grains, and has a void ratio in the densest state of about 0.61 and in the loosest state of about 0.88.

[53] "A Carrier Type Strain Indicator," by George W. Cook, *Report No. 565*, David Taylor Model Basin, U. S. Navy, Washington, D. C., November, 1946.

(*2*) *Cambridge* (*Mass.*) *Clay*.—This is a medium soft, inorganic clay, with occasional thin silt partings, brittle in the undisturbed state and soft and sticky when remolded. Its natural water content ranges from 30% to 50%. For layers of this clay having a natural water content of from 40% to 50%, the liquid limit was found to range from 44 to 59 and the plastic limit from 21 to 27. For layers of this clay having a natural water content of from 30% to 40%, the liquid limit was found to range from 37 to 44 and the plastic limit from 20 to 23.

(*3*) *Boston* (*Mass.*) *Clay*.—This is similar, geologically and in its appearance, to the Cambridge clay. The samples tested had a natural water content between 32% and 36%, an average liquid limit of 42 and a plastic limit of 20.

(*4*) *Stockton* (*Calif.*) *Clay*.—This is a tough, brown clay, locally called adobe. The sample tested was obtained from the compacted fill on which the Stockton pavement traffic test[54] was conducted, and was about 90% saturated, with a natural water content of about 25%, a liquid limit of from 60 to 64, and a plastic limit of from 20 to 23.

(*5*) *Atlantic Muck* (*Canal Zone*).—This is an organic clay, having a natural water content ranging between 50% and 135%, a liquid limit of from 55 to 95, and a plastic limit of from 30 to 55. The samples tested were obtained in 3-in. diameter, thin-walled tubes from a boring into a natural deposit located adjacent to the Panama Canal.

(*6*) *Cucaracha Shale* (*Canal Zone*).—This is a slickensided clay-shale. The samples tested were obtained from core borings in the Cucaracha formation located in the vicinity of the Gaillard Cut, Panama Canal.

Details of Testing.—The Cambridge and Boston clays were tested both in unconfined compression and in triaxial compression, whereas the Stockton clay was tested in triaxial compression only. Cucaracha specimens were tested in triaxial compression and Atlantic muck in unconfined compression. Manchester sand was tested in the dense state, dry, in a vacuum-type triaxial compression apparatus.

Most unconfined compression test specimens of Cambridge clay were 6.3 cm square and about 16 cm high; all triaxial compression test specimens of clay were 3.56 cm in diameter and about 9 cm high. Unconfined compression test specimens of Atlantic muck were 2.5 cm square and about 6 cm high. Triaxial compression test specimens of Cucaracha shale were 5 cm in diameter and 12 cm high, and specimens of Manchester sand were 7.1 cm in diameter and about 18 cm high.

Two types of triaxial compression tests were performed, designated "quick" and "consolidated-quick." A quick triaxial compression test is one in which there is no preliminary consolidation and no drainage of pore water during the test. A consolidated-quick triaxial compression test is one in which the specimen is allowed to consolidate under a hydrostatic pressure, but, during the subsequent quick axial loading, there is practically no drainage of pore water. A detailed discussion of these types of tests appears elsewhere.[55]

[54] "Flexible Pavement Test Section for 300,000-Lb. Airplanes, Stockton, California," by Ralph A. Freeman and O. J. Porter, *Proceedings*, Highway Research Board, National Research Council, Vol. 25, 1945, pp. 23–44.

[55] "Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils," U. S. Waterways Experiment Station, Vicksburg, Miss., April, 1947.

The triaxial compression test specimens were surrounded by two rubber membranes, each averaging approximately 0.05 mm thick. Specimens of clay and Cucaracha shale for consolidated-quick triaxial compression tests were fully consolidated under a lateral pressure of either 3 kg per cm^2 or 6 kg per cm^2 before axial compression was started. Manchester sand specimens were tested under a lateral pressure of 0.3 kg per cm^2 and 0.9 kg per cm^2.

Two procedures for performing transient compression tests were used, which are described below:

Controlled Impulse Method.—The specimen is subjected to a controlled impulse. The peak load exerted on the test specimen and its deformation depend on the stress-deformation and strength characteristics of the specimen. Typical stress-time and strain-time diagrams for an unconfined transient compression test on clay, using the controlled impulse method, are shown in Fig. 82.

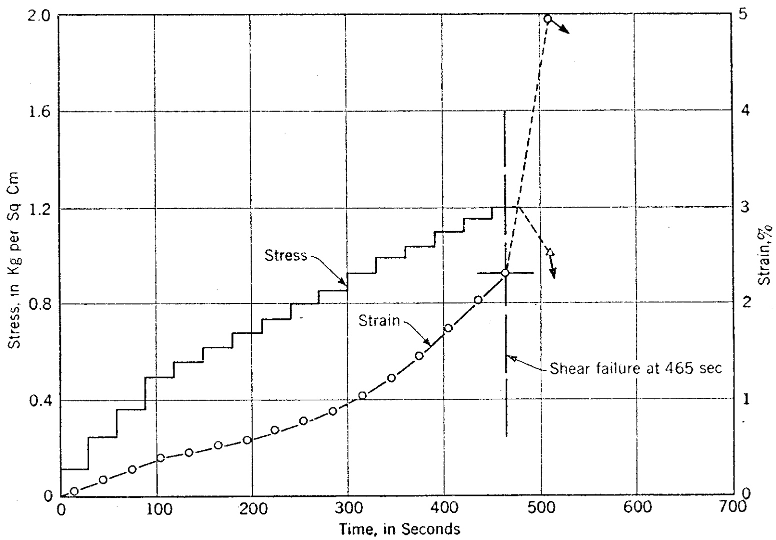


Fig. 82.—Time Curves for Stress and Strain; Unconfined Transient Compression Test on Cambridge Clay

Controlled Strain Method.—The specimen is subjected to a rate of strain which is maintained approximately constant from the beginning of the test to the desired maximum strain. The peak load exerted on the specimen and the time of loading depend on the stress-deformation and strength characteristics of the specimen.

Results of Tests

Representative results of a transient and a static unconfined compression test on Cambridge clay are shown in Figs. 82, 83, and 84. Figs. 82 and 83 are time curves for stress and strain and Fig. 84 shows the stress-strain curves. (The plotted points shown in Fig. 83 were taken from a continuous oscillograph record.)

All unconfined compression specimens of Cambridge clay failed with one or more clearly visible shear planes. To measure the slope of the shear planes, seven transient tests, with a time of loading of about 0.02 sec, and four static tests were so conducted that loading was stopped as soon after the shear planes

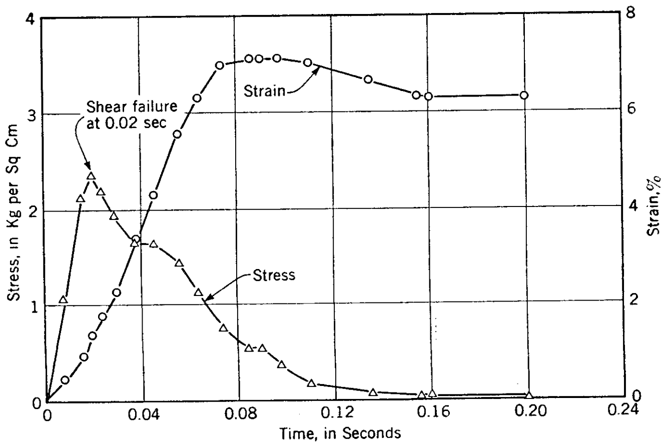


FIG. 83.—TIME CURVES FOR STRESS AND STRAIN; UNCONFINED STATIC COMPRESSION TEST ON CAMBRIDGE CLAY

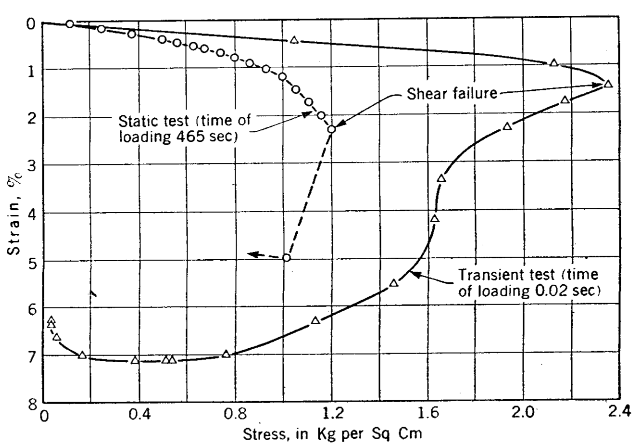


FIG. 84.—STRESS-STRAIN CURVES; UNCONFINED COMPRESSION TESTS ON CAMBRIDGE CLAY

appeared as was technically possible. The strain at the end of these tests ranged between 2% and 3% and it is believed that the distortion of the shear planes in these tests was negligible. The angles between the shear planes and the horizontal (plane of major principal stress) were carefully measured to determine whether there was a significant difference in these angles between static and fast transient loading. For the static tests, the angles ranged between 53°

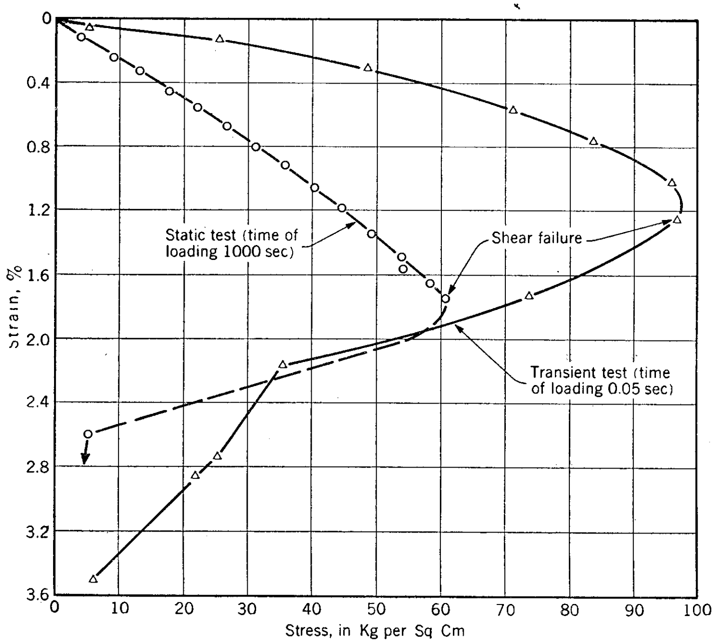


FIG. 85.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC TRIAXIAL COMPRESSION TESTS (CONSOLIDATED-QUICK) ON CUCARACHA SHALE (σ_3 = 6 KG PER SQ CM)

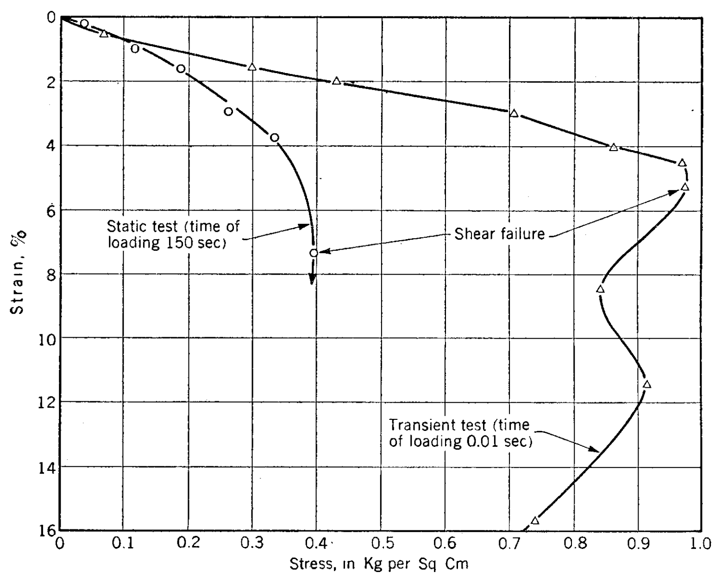


FIG. 86.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC UNCONFINED COMPRESSION TESTS ON ATLANTIC MUCK

and 68°, with an average of 61°; and for the transient tests, between 47° and 65°, with an average of 58°.

Stress-strain curves for a transient and a static triaxial compression test on Cucaracha shale are shown in Fig. 85. This material failed along one or more clearly defined shear planes sloping from 35° to 65°. Usually the position

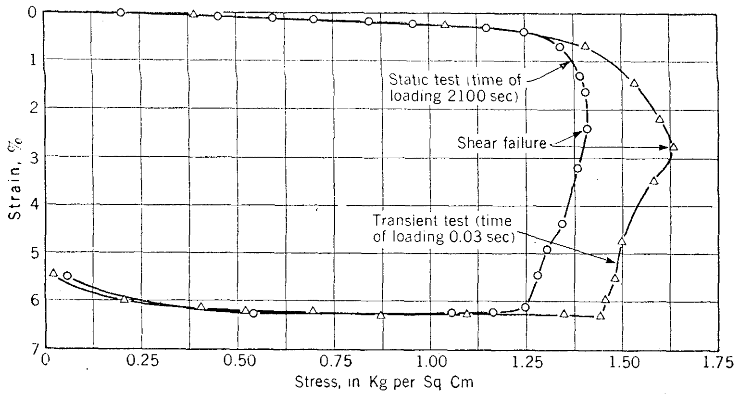


FIG. 87.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC TRIAXIAL COMPRESSION TESTS ON MANCHESTER SAND (σ_3 = 0.3 KG PER SQ CM)

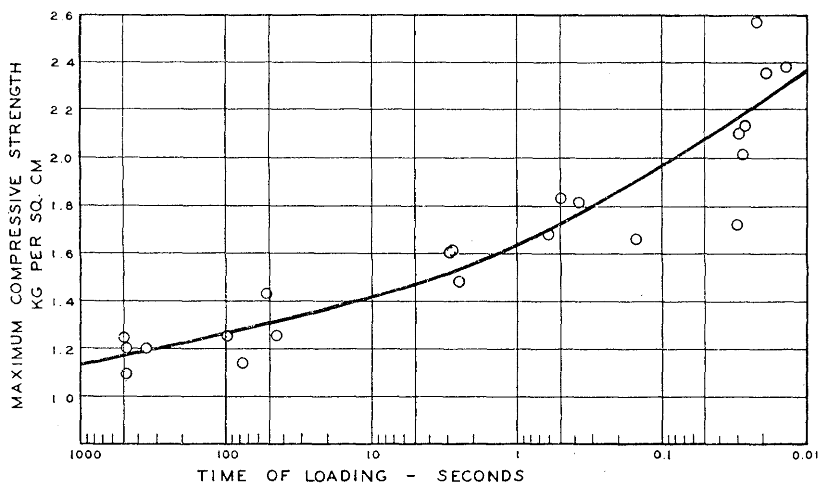


FIG. 88.—UNCONFINED COMPRESSION TESTS ON CAMBRIDGE CLAY

and slope of the shear planes were influenced by jointing and the presence of weaker phases of the Cucaracha shale. Both specimens tested represent a strong phase of Cucaracha shale. The points shown for the transient test were taken from a continuous oscillograph record.

Stress-strain curves for a transient and a static unconfined compression test on Atlantic muck are shown in Fig. 86. This material usually failed along one or more clearly defined shear planes sloping about 50°. Some specimens failed by bulging with vertical tension cracks.

Stress-strain curves for a transient and a static vacuum-type triaxial compression test on Manchester sand are shown in Fig. 87. Both of these tests

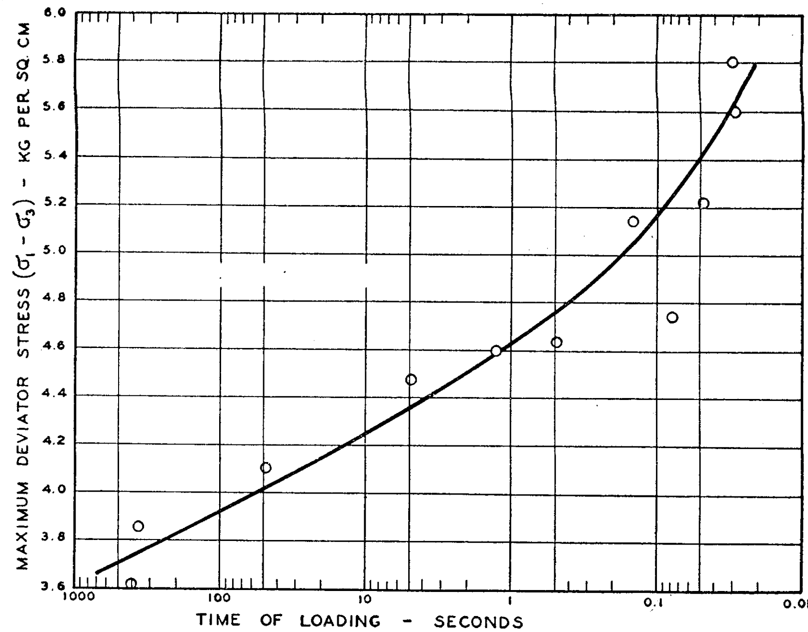


FIG. 89.—TRIAXIAL COMPRESSION TESTS ON CAMBRIDGE CLAY (σ_3 = 6 KG PER SQ CM)

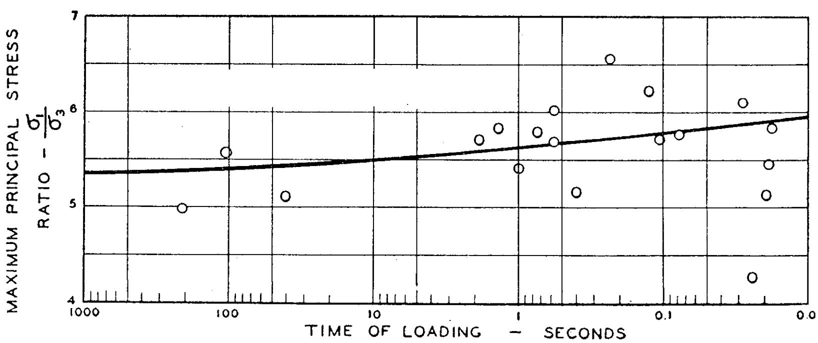


FIG. 90.—TRIAXIAL COMPRESSION TESTS ON SAND (VOID RATIO = e_p = 0.620; VACUUM PRESSURE $-P_V$ = 0.30 KG PER SQ CM)

were performed in a vacuum-type triaxial compression apparatus on dense sand (e = 0.61). The points shown for the transient test were taken from the continuous oscillograph record. Failure of all specimens was by bulging without noticeable shear planes.

The relation between compressive strength and time of loading for a given material is determined from a series of static and transient compression tests. Such relationships are shown by Figs. 88 and 89 for Cambridge clay, and by Fig. 90 for Manchester sand.

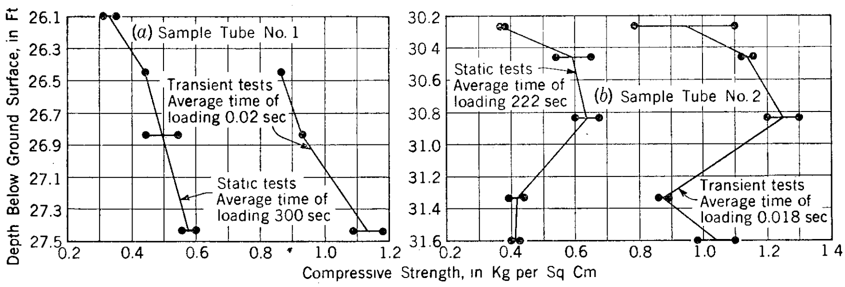


FIG. 91.—COMPRESSIVE STRENGTH VERSUS TIME OF LOADING FOR ALL TEST SERIES ON CLAY

Fig. 91 is a summary sheet showing the relationships between compressive strength and time of loading as obtained from all the tests on clays performed to the time this paper was prepared. An explanatory legend for Fig. 91 is given in Table 31.

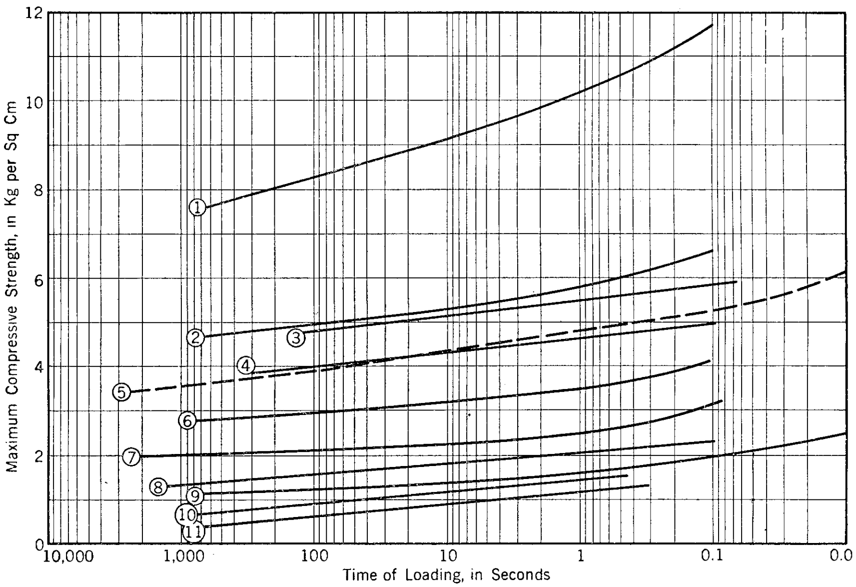
TABLE 31.—LEGEND FOR FIG. 91

Curve No.	Sample No.	Source of clay[a]	Type of test	Water content at time of test[b] (%)
1	HP-6-C	Cambridge clay which was slowly dried to slightly higher than its shrinkage limit	Unconfined compression	27
2	HP-6-C	Cambridge clay which was slowly dried from its natural water content of about 47% to about 33%	Unconfined compression	33
3	HP-6-B	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm^2	24 to 32
4	HP-6-A	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm^2	32 to 35
5	HP-6-D	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm^2	40 to 44
6	HP-6-C	Cambridge clay which was slowly dried from its natural water content of about 47% to about 37%	Unconfined compression	37
7	HP-7-A	Stockton clay	Quick triaxial compression $\sigma_3 = 3$ kg per cm^2	24 to 27
8	HP-5-A	Boston clay	Quick triaxial compression $\sigma_3 = 6$ kg per cm^2	32 to 35
9	HP-2	Cambridge clay	Unconfined compression	34 to 39
10	HP-6-B	Cambridge clay	Unconfined compression	35 to 40
11	HP-6-A	Cambridge clay	Unconfined compression	40 to 42

[a] All specimens were tested undisturbed. [b] For curves Nos. 1 to 6, the specimens were reduced in water content from their original by consolidation or drying. For curves Nos. 7 to 11, the specimens were tested at their original water content.

For the weakest clays the transient strength for the fastest time of loading was about twice the 10-min static strength. The ratio between these two values appears to decrease with increasing strength of the material. For the strongest clays included in Fig. 91, the fast transient tests showed a strength that was only about 50% greater than the 10-min static strength. Sufficient data are not available to plot the corresponding relationships for Cucaracha shale and Atlantic muck.

The stress-strain curves for Cucaracha shale in Fig. 85 indicate that the strength in a transient test, with a time of loading of 0.05 sec, is about 60% greater than that for a 10-min static test.



Sample tube No.	Average Time of Loading (Sec)		Average Modulus of Deformation (Kg per Sq Cm)	
	Static tests	Transient tests	Static tests	Transient tests
1	300	0.02	8.1	16.0
2	222	0.019	8.7	19.4

Fig. 92.—Unconfined Compression Tests on Atlantic Muck

Results of unconfined compression tests on Atlantic muck are summarized in Table 32. Fig. 92 shows profiles of results for two sample tubes. These tests indicate that the transient strength for a time of loading of about 0.02 sec is about twice the 10-min static strength.

Modulus of Deformation.—The original purpose of the investigation did not include the determination of the stress-deformation characteristics. Therefore, precise measurements of the deformation at small loads in transient tests have not yet been made. However, all tests have been analyzed as well as possible for the purpose of arriving also at some information regarding the effect of time of loading on the initial slope of the stress-deformation curve as represented by the modulus of deformation. For the purpose of this investigation, the modulus of deformation is defined as the slope of a line drawn from the

TABLE 32.—RESULTS OF UNCONFINED COMPRESSION TESTS ON ATLANTIC MUCK

Tube No.	Depth (ft) below ground surface	Water content[a] (%)	Static compressive strength[b] (kg per cm²)	Transient compressive strength[c] (kg per cm²)
1	24.1 to 28.1	109	0.31 0.35	....[d]
		78	0.44	0.87
		77	0.54 0.44	0.94
		85	0.55 0.60	1.18 1.09
2	28.1 to 32.1	67	0.38 0.37	0.69 1.00
		63	0.54 0.65	1.12 1.16
		59	0.60 0.67	1.30 1.20
		60	0.44 0.39	0.89 0.86
		52	0.40 0.43	0.98 1.10
4	35.4 to 39.2	120	0.33 0.33	....[d]
		118	0.26 0.20	....[d]
		121	0.37 0.31	0.64 0.91
		123	0.40 0.43	1.00 0.95
		122	0.47 0.53	....[d]
6	43.1 to 46.8	114	0.18 0.18	....[d]
		134	0.42 0.45	0.89 0.55
		135	0.43 0.34	0.68 0.88

[a] Water contents are average values for the static and transient tests for which the strength values are shown in the same horizontal column. [b] Time of loading ranging between 3 min and 9 min. [c] Time of loading approximately 0.02 sec. [d] Recording instruments failed to function satisfactorily.

origin through the point on the stress-deformation curve corresponding to a stress of one half of the strength.

The order of magnitude of the modulus of deformation, as obtained from the tests performed on clay and Cucaracha shale samples, is summarized in Table 33.

From Table 33, it is concluded tentatively that, for these cohesive materials, the modulus of defromation for fast transient loading is about twice the value for 10-min static loading.

TABLE 33.—MODULUS OF DEFORMATION OF CLAYS AND CUCARACHA SHALE

Material	Type of test	AVERAGE VALUES OF MODULUS OF DEFORMATION (KG PER CM²)	
		Static tests	Fast transient tests
Cambridge clay.....	Unconfined compression	200	400
	Consolidated-quick triaxial (σ_3 = 6 kg per cm²)	650	1,300
Boston clay.........	Quick triaxial	250	500
Stockton clay.......	Quick triaxial	250	500
Atlantic muck......	Unconfined compression	9	18
Cucaracha shale.....	Consolidated-quick triaxial (σ_3 = 6 kg per cm²)	5,000	12,000

As can be noted in Figs. 82 to 86, the slope of the stress-strain curves for transient tests appears to be almost constant to 50% of the failure load. Hence, the order of magnitude of the modulus of deformation for transient tests is independent of the stress, provided a reference stress of not more than one half of the compressive strength is selected. For this reason the ratio of the moduli for transient and static tests would remain about the same, even if the reference stress for a transient test is assumed equal to one half of the static strength.

The modulus of deformation of dry Manchester sand had the same order of magnitude in transient tests as in static tests, as shown by the nearly identical slopes of the stress-strain curves, Fig. 87.

CONCLUSIONS

The findings presented in this paper can be summarized as follows:

(1) The strength of the clays and the Cucaracha shale loaded to failure in about 0.02 sec was found to be between 1.5 and 2.0 times greater than their 10-min static strengths.

(2) The strength of sand increases only slightly with decreasing time of loading. The strength for the fastest time of loading (0.02 sec) was about 10% greater than that for 10-min static tests.

(3) The modulus of deformation of clay and Cucaracha shale for fast transient tests (time of loading about 0.02 sec) was found to be approximately twice that for 10-min static tests.

(4) The modulus of deformation of sand was found to be independent of the time of loading.

ACKNOWLEDGMENT

The writers wish to express their appreciation to Colonel Stratton for initiating this investigation and for his encouragement. The following members of the research staff of the Harvard Soil Mechanics Laboratory, who were engaged on this project, contributed to its success by valuable suggestions: L. S. Chen, J. V. Grasso, H. B. Sutherland, Edward Wenk, Jr, Jun. ASCE, and S. D. Wilson. The David Taylor Model Basin, United States Navy, assisted materially on questions of instrumentation and by the loan of strain indicators and recording instruments.

In connection with the use of the SR-4 strain gages, advice was obtained from A. C. Ruge, M. ASCE. The hydraulic loading apparatus was largely designed by Lessells and Associates, Consulting Engineers.

CONSTRUCTION PLANNING AND METHODS

BY J. J. ROSE,[56] ESQ., F. L. DYE,[57] M. ASCE, W. B. WATSON,[58] ESQ., AND L. T. CROOK,[59] JUN. ASCE

SYNOPSIS

The principal construction features of the proposed Panama sea-level canal are described in this paper, including the various plans for the conversion of existing lock canal to sea level, and the construction methods on which the cost estimates are based.

Conversion of the existing canal to sea level in a single stage by deep dredging is preferable to conversion by stage dredging because it would be cheaper, would require a shorter construction period, and would interfere less with canal traffic.

For purposes of construction planning and estimates, the bulk of the dry excavation would be performed by large shovels or draglines of at least 25-cu-yd capacity, loading directly into dump scows which would haul the spoil to disposal areas in Gatun Lake. The wet excavation would be performed by conventional dredges supplemented by special hydraulic and bucket-ladder dredges capable of dredging to a depth of 145 ft below water surface. These methods of dry excavation and wet excavation are the most feasible and economical, and can be adapted most readily to the over-all project.

INTRODUCTION

Public Law No. 280 which authorized the Isthmian Canal Studies—1947 directed "a comprehensive review and study, with approximate estimates of costs, of means of increasing the capacity and security of the Panama Canal * * * [and] study [of] * * * a canal or canals at other locations * * *."

Compliance with Public Law No. 280 required initially the following studies:

1. Selection of general construction techniques and estimating methods to obtain approximate and comparable estimates of cost for the many canal routes to select the routes offering the best possibilities for development; and
2. Determination of the most practicable construction procedures and equipment, and preparation of detailed and dependable comparative estimates of costs of the best canal plans, to assist in the final selection of route and type of canal.

These procedures led to the selection of the Panama sea-level canal as the most economical means of increasing capacity and security to meet the future needs on interoceanic commerce and national defense. After the selection of

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the sea-level canal at Panama, more detailed construction studies and estimates were made. The main features of these studies were:

(1) Investigation of new and special equipment for converting the Panama Canal to sea level; and

(2) Investigation of the major construction problems involved in executing the recommended plan of converting the present Panama Canal to sea level in a single stage by the "deep-dredging" method.

For the initial construction studies under Public Law No. 280, thirty possible routes or alinements (Fig. 6) were examined, of which eight were found on map inspection and, in some cases, on field reconnaissance, to offer the best possibilities for development. Cost estimates for the more favorable of the sea-level and lock canals on eight routes were prepared by using excavation quantities obtained from center-line map profiles and, in each case, approximate classifications of materials. Detailed investigations were made of the problems involved in the construction of both a lock canal and a sea-level canal in the Canal Zone and the immediate vicinity, and estimates of costs based on standard construction procedures were prepared for use in comparative analyses. Unit costs for excavation were those developed for the Panama route. Costs of structures, such as locks, were developed for each route on the basis of height or lift, using the cost of locks at Panama as the estimating standard. As shown by Tables 1 and 2, the Panama route is the most economical for both a sea-level canal and a lock canal.

After selection of the Panama sea-level canal, detailed studies and cost estimates were made of the most efficient and economic construction methods. The investigations and studies used as the basis for the estimates of cost for the construction of the Panama sea-level canal are described in this paper.

Features of the Main Channel

The Panama sea-level canal would have a channel 600 ft wide at a 40-ft depth, with a total depth below mean sea level varying uniformly from 60 ft at the Atlantic entrance to 70 ft at the Pacific end. The principal features of the canal are shown in Fig. 7. The total length of the canal from deep water in the Atlantic to deep water in the Pacific would be approximately 46 miles. The alinement would follow, generally, that of the present canal. Sharp curves would be eliminated and tangents lengthened to meet navigation requirements. The sea-level canal alinement would be contiguous to the present canal in the section of deep cut through the Continental Divide and would permit the removal of a large part of the required rock excavation more economically by dry methods. The alinement would utilize the excavation accomplished for the Third Locks structures and the approach channels at Miraflores and Gatun. The total required excavation is approximately 1,070,000,000 cu yd, of which 750,000,000 cu yd could be excavated in the dry. Dredging would be required for the removal of the remaining 320,000,000 cu yd. For comparison, the channel excavation by the United States to complete the existing canal totaled approximately 120,000,000 cu yd of wet and 125,000,000 cu yd of dry. The total usable channel excavation, including that done by the French, was approximately 275,000,000 cu yd.

The maximum depth of cut to be excavated would be 650 ft; however, the length of canal at this depth would be less than ½ mile. Approximately 3½ miles of canal would be in cut deeper than 300 ft; 7 miles, in cut between 200 ft and 300 ft; 9 miles, in cut between 100 ft and 200 ft; and the remaining 26 miles, in cut less than 100 ft deep. Of this 26 miles, 10 miles would be in a muck formation, a soft overburden material in which the maximum depth of cut would be less than 80 ft.

For convenience in estimating and for facilitating orderly completion of the excavation, the channel excavation has been divided into ten parts. The material to be excavated has been classified into the following four types according to the relative difficulties of excavation: Common, soft rock, medium rock, and hard rock, as shown in Table 34. The linear distribution of the material is shown in Fig. 93.

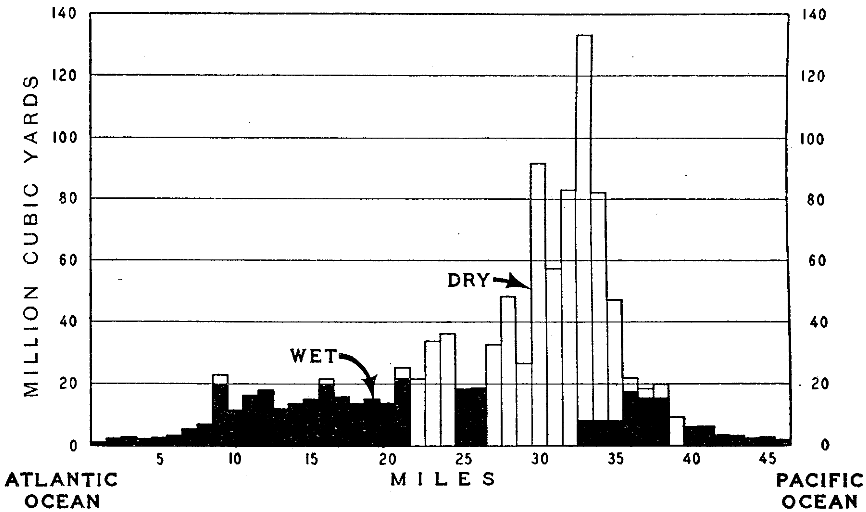

FIG. 93.—DISTRIBUTION OF EXCAVATION QUANTITIES

STRUCTURAL FEATURES

Tidal-Regulating Structures.—These structures consist of a tidal lock, a navigable pass, and a gated water-control structure, and involve 52,000,000 cu yd of excavation and 1,800,000 cu yd of concrete. The construction of these structures would involve only conventional methods.

Flood-Control Structures.—The flood-control structures consist principally of earth dikes and dams, constructed mainly with excavation spoil. Construction of the flood-control structures except as a method of spoil utilization is conventional.

Housing and Facilities for Construction Workers.—The Canal Zone is a government-administered area in which, with rare exceptions, there is no private industry, business, or ownership of land or buildings. Consequently,

TABLE 34.—Excavation Classification; Canal Zone Rock Units

Formation	Description	Unit weight (lb per cu ft)	Quantity (Thousands of Cu Yd)		Dynamite (lb per cu yd), dry excavation	Dynamite (lb per cu yd), wet excavation
			Dry	Wet		
(a) Common Excavation						
Overburden and muck....	Muck, silts, sands, clays, gravels, etc.	Variable	108,439	165,044	0	0
(b) Soft Rock						
Cucaracha..............	Largely dense greenish-gray clay shales highly slickensided within certain horizons. Black carbonaceous shales, sandstones, and conglomerates in subordinate proportions	135–140	125,584	27,406	0.30 (40%)	0.65 (40%)
Culebra................	Medium-hard sandstones, soft sandy and carbonaceous shales	140±	146,647	8,216	0.30 (40%)	0.65 (40%)
La Boca................	Dense silty or sandy dark-gray shales with intercalated sandstone beds sporadically present	140±	17,993	3,572	0.30 (40%)	0.65 (40%)
Gatun..................	Fine-grained argillaceous and calcareous sandstones with interbedded dense tuffs and conglomerates	120–125	14,330	7,409	0.30 (40%)	0.65 (40%)
(c) Medium Rock						
Caimito................	Coarsely bedded medium-grained and fine-grained medium-hard limy sandstones and tuffs	130±	94,497	47,236	0.50 (40%)	1.00 (60%)
Las Cascadas...........	Agglomeratic tuff and tuff-breccia consisting of angular fragments of hard dark-gray andesite in a clayey dark-gray to light-green altered tuff matrix	140±	13,778	0	0.50 (40%)	1.00 (60%)
Bohio conglomerate.....	Subangular to rounded pebbles, cobbles, and boulders up to 2 ft in diameter in a dark-gray or brown generally coarse friable tuffaceous sand matrix	145±	81,722	50,463	0.50 (40%)	1.00 (60%)
(d) Hard Rock						
Pedro Miguel and Bas Obispo agglomerate....	Hard light-gray to dark-gray fine-textured to coarse-textured agglomerates and tuffs	155±	64,645	1,884	0.75 (60%)	1.50 (60%)
Basalt.................	Hard columnar-jointed basalt flows and intrusives	160–170	82,288	7,516	0.75 (60%)	1.50 (60%)
Total..............	....	....	749,923	318,746	....	....

housing, water supply, sewage disposal, power, communications, hospital and health facilities, schools, fire and police protection, and other facilities would have to be provided for the expanded population of the Canal Zone during canal construction. Housing and other facilities would be largely temporary in character but certain permanent construction would be performed to replace existing housing and other facilities which date back to the early 1900's and which are, or soon will be, obsolete.

Program

Various programs of construction were evaluated, and it was determined that a 10-year construction program to convert the existing canal to sea level would be the most favorable. A shorter program would result in less economical use of special plant for dry and wet excavation and would require more housing. A longer program would probably result in somewhat lesser costs, but would delay the fulfilment of the requirements of national defense.

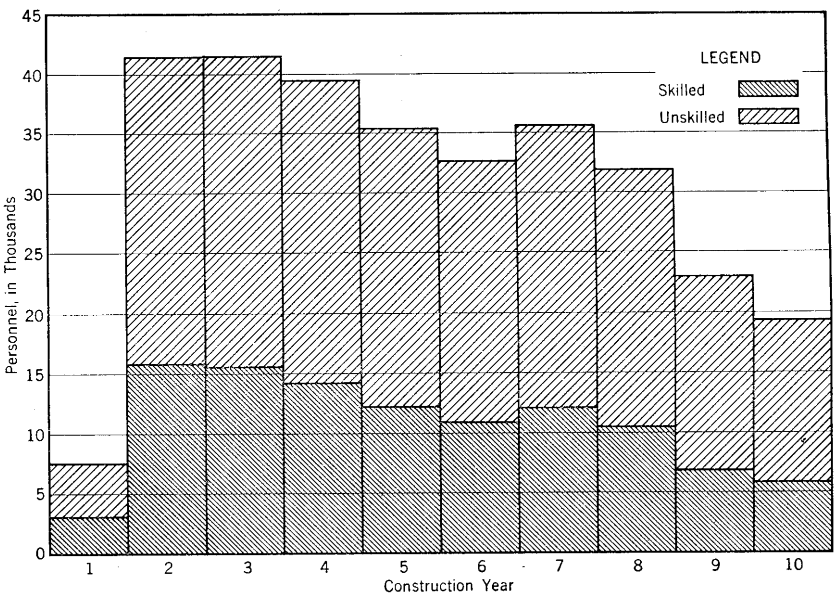

FIG. 94.—TOTAL PERSONNEL REQUIREMENTS, PANAMA SEA-LEVEL CANAL

Personnel

Skilled and technical personnel would be obtained from the United States. The unskilled employees would be largely indigenous to the Caribbean area. Contracts with skilled and unskilled labor would include payment of transportation from the place of recruitment and return. Transportation of government workers' families and household furnishings would be provided. The labor requirements are shown in Fig. 94.

Materials

Satisfactory concrete aggregates may be obtained from alluvial deposits of sand and gravel in the Chagres River or from basaltic rocks from the channel excavation. Local sources for cement, steel, and lumber are not adequate, and these items would be obtained from the United States. The large volumes of excavation throughout the entire length of canal and water transportation make available adequate and economical embankment materials for earth dam construction.

Conversion to Sea-Level Canal by Stage Dredging

The method of converting the present canal to sea level has a direct effect on the method of excavation; and, in turn, the various possibilities of performing the excavation have a direct bearing on the selection of the method of conversion.

In all previous studies of plans for effecting the conversion of the present canal, it was contemplated that Gatun Lake would be lowered by stages. In one study, seven stages of lowering were planned; in others, three stages were selected. Stage lowering would require progressive alteration of the existing locks so that traffic could be accepted at all stages of the lake. This program would involve certain risks to shipping and to the canal if auxiliary conversion locks were not built, since only one lane of locks would be open to traffic at a time while the other was being modified to accept traffic at the next lower stage.

In the current studies it was found that stage conversion could best be effected by lowering the lake in three stages—from El. 85 to El. 54 (the elevation of Miraflores Lake), from El. 54 to El. 22, and from El. 22 to sea level. For this plan of conversion, equipment would be required capable of dredging to depths of 72 ft, a not uncommon depth of excavation. It was concluded that the risk in depending on one lane of locks during conversion was not warranted and that continuous operation of two lanes of locks would be essential. Consideration was first given to the use of a third lane of locks, as planned in earlier studies, constructed to full summit height (El. 85) to overcome this serious handicap to the stage conversion plan. However, it was determined later that the three-stage lowering plan made it practicable and economical to construct the special two-lane, single-lift conversion locks, to El. 54, at Miraflores and at Gatun. The conversion locks would have a lower lift and would be no more costly than would the alternate plan involving a third single-lane, high-lift lock at each end of the existing lock sites and the conversion of the existing locks. The conversion locks would have upper sills at a low elevation to permit the transit of ships at each successive stage of lowering below El. 54. In this plan the existing locks would be abandoned when Gatun Lake was lowered to El. 54, and the temporary conversion locks at Gatun and Miraflores would then be placed in operation.

The disadvantages of the stage conversion plan are:

a. Conversion locks would be required and would add considerably to the cost of the project.

b. All dry excavation, approximately 750,000,000 cu yd of the total of more than 1,000,000,000 cu yd, would have to be excavated prior to the initial lowering of Gatun Lake (thus requiring a minimum construction program of 15 years to permit sufficient time for the two remaining stages of dredging).

c. Traffic in the canal would be interrupted during the drawdown for each stage.

d. Part of the material dredged during the second stage and all the material dredged during the final stage would require locking to sea for disposal, because the drawdown would prevent using Gatun Lake as a spoil area, and use of the locks for this purpose would interfere with canal traffic. (Approximately 55,000,000 cu yd would have to be transported to sea.)

e. The lowering of Gatun Lake for the final stages would deplete the storage area to such an extent that pumping of lockage water would be required.

Single-Stage Conversion by Deep Dredging

A proposal by E. E. Abbott for conversion by deep dredging led to an investigation of the possibilities of adapting or developing dredging equipment so that all wet excavation could be performed in advance of the lowering of the summit lake. This investigation indicated that such a method would be entirely feasible. Drilling, blasting, and excavating to depths of 145 ft below the surface of Gatun Lake would be required to provide the design depth in a sea-level canal.

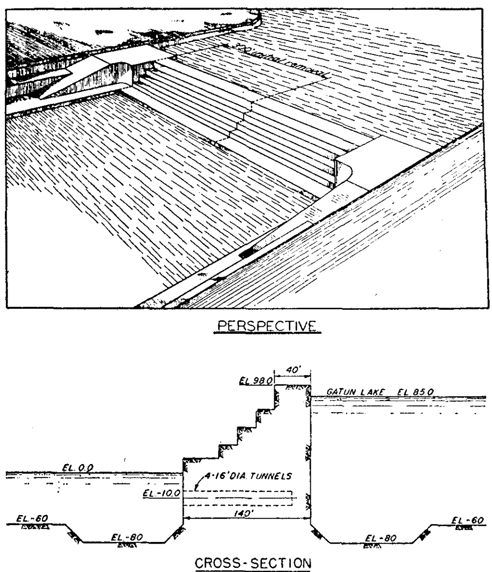

FIG. 95.—GATUN CONVERSION PLUG

In this plan, the present operating levels of Gatun Lake (El. 85) and Miraflores Lake (El. 54) would be maintained during the entire period of excavation of the canal. The level of Gatun Lake would be dropped to El. 81 a short time prior to final lowering to permit placing the flood-control system of the canal in operation. The canal would then be closed to traffic for about 7 days, during which the lakes would be lowered to sea level and the channel would be cleared of the barriers used to retain Gatun and Miraflores lakes during excavation.

The barriers or channel plugs used to retain the lakes would consist of the following: (1) A natural rock barrier across the sea-level channel on the north shore of Gatun Lake (Fig. 95); (2) a natural rock barrier in the sea-level channel separating Gatun and Miraflores lakes near the existing Pedro Miguel Locks (Fig. 96); and (3) a temporary steel dam in the north approach to the tidal lock (Fig. 97). The natural rock plugs at Gatun and Pedro Miguel would be removed progressively by blasting and dredging during the 7 days that Gatun Lake was being lowered. The temporary steel dam in the approach to the tidal lock would be removed in sections by derricks when the water in the canal reached sea level.

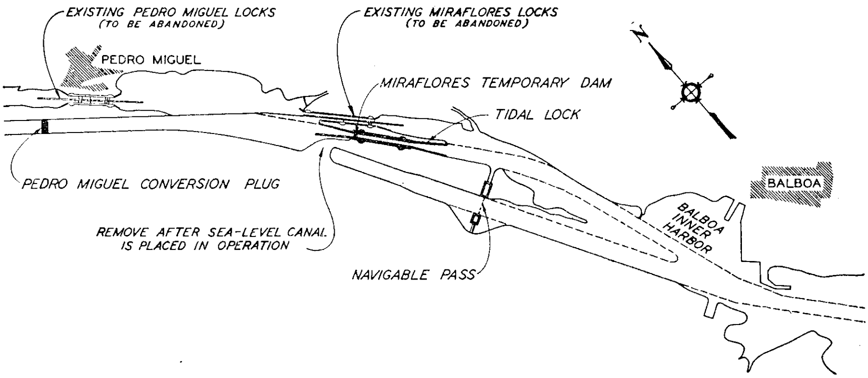

FIG. 96.—LOCATION PLAN, PEDRO MIGUEL PLUG AND MIRAFLORES TEMPORARY DAM

Gatun Conversion Plug.—The Gatun conversion plug would be of sandstone (Fig. 95). The bedding of this formation is massive, and the infrequent near-vertical joint planes are tight. The lake approach to the Gatun plug would be excavated by deep-dredging methods, but the immediate face of the plug would be carefully formed by line drilling and light blasting. The Atlantic approach would be excavated by dry methods. Four discharge outlets, 16 ft in diameter, would be tunneled in the plug from the downstream side of the plug to within about 20 ft of the face. The entire mass of the rock plug would be prepared for blasting and final removal by drilling vertical holes in a 5-ft pattern down to El. −50. Initially, the lake would be lowered through the flood-control spillways and the lock culverts. Later the tunnel outlets in the rock plug would be opened by blasting, using drill holes from the top of the plug for loading and firing, and finally the entire remaining mass of the plug would be blasted.

Pedro Miguel Conversion Plug.—The Pedro Miguel conversion plug would be generally similar to that at Gatun. It would be located near the upper approach to the existing Pedro Miguel Locks (Fig. 96). It would be placed as close to Miraflores Lake as sound rock permits, to reduce the volume of channel excavation required to be transported through the Miraflores Locks for disposal at sea. The agglomerate of which this plug is composed is a relatively impervious rock massively jointed. Any leakage through material of this type would occur along joints or fracture planes, and the condition would become evident during the progress of excavation. Grouting and sealing of the upstream side would be undertaken if found necessary. During the lowering of the lake, the plug would be removed progressively in a manner similar to that used at Gatun.

Miraflores Temporary Dam.—The upstream arm of the construction cofferdam, enclosing the tidal lock, was considered for use as the plug for Miraflores Lake. The removal of the cofferdam and the excavation of the channel under the cofferdam that could not be done until drawdown of the lake had been completed would not be accomplished as rapidly as the removal of the Gatun and Miraflores plugs, and traffic would be suspended for a longer period. Therefore, a temporary steel dam is planned in the north approach to the tidal lock, illustrated in Fig. 97, which would be capable of holding Miraflores

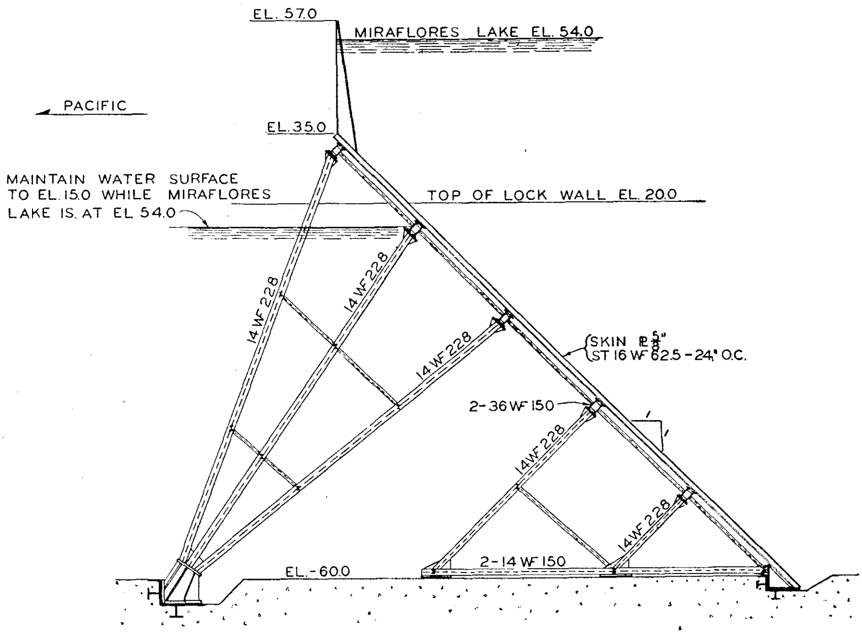

FIG. 97.—CROSS SECTION, TEMPORARY STEEL DAM IN TIDAL LOCK

Lake at its present elevation. It would be constructed in the completed lock approach above the upper gates prior to the removal of the construction cofferdam. Short sections of earth embankments extending to high ground, with temporary concrete-retaining abutments on the lock walls, would be required to complete the closure. The dam would be constructed in a manner permitting rapid disconnection and removal of individual sections in 1 day by two derricks mounted on the lock walls.

Advantages of Deep-Dredging Plan.—The principal advantage of the deep-dredging method of excavation over the stage-dredging plan is in the saving of lock conversion costs. Also, the deep-dredging plan permits full use of the Gatun Lake area for deposit of all excavation spoil, with corresponding saving in excavation costs. Using Gatun Lake for the hauling of excavated materials would make it possible to construct the Caño Quebrado, Trinidad, and Monte Lirio flood-control dams with greater facility and at less cost. This plan would also result in less interference with canal traffic and would save about 5 years in construction time.

The net saving of the deep-dredging plan over the stage-dredging plan is estimated at $130,000,000.

Dry Excavation

Conversion of the present Panama lock canal to a sea-level canal would require the excavation in the dry of 750,000,000 cu yd of material, practically all being between miles 21 and 24 and miles 27 and 36, Fig. 93. Considering the character and the volume of material to be removed, the nature of the terrain in this area, and the accessibility of spoil areas, it is evident that various types of equipment could be employed in the excavation and removal of material.

Operation Phases.—The most suitable method of excavation is that which has the widest general application and which results in the least cost for removal and disposal of the excavated material. It can be selected only by an examination of the principal phases of the excavation operations, namely: (*a*) Preparation of the material for excavation; (*b*) excavation; and (*c*) haul of excavated material to disposal areas.

(*a*) Preparation of the Material for Excavation.—For common material no preparation is required, and for soft rock preparation may or may not be required, depending on the selection of excavation equipment. For medium and hard rocks, preparation requires systematic drilling and blasting, the extent of which is determined by the character of the material and the type and and size of excavation equipment selected.

(*b*) Excavation.—A wide choice of equipment of various types and sizes is available. The utility of the different types of equipment depends on the character of material, the nature of its occurrence, and other factors discussed more fully elsewhere in this paper.

(*c*) Haul of Excavated Material to Disposal Areas.—This factor, because of the large quantity of materials involved, is of major, if not controlling, importance in the selection of the method of dry excavation. Accordingly, the methods of haul are discussed first.

Haul.—Haul refers to the transportation of the material from the place where it is excavated to the point of disposal—by truck, railroad, belt conveyer, scraper, scow, or other methods. The selection of the method is determined by the character of the excavation and its location in relation to the disposal area, the gradient between the two points, the location for suitable haul roads or conveyer lines—and in the case of scow haul by the availability of water transportation between the point of excavation and the disposal area.

Because the alinement of the proposed Panama sea-level canal takes advantage of the low terrain, the available dry spoil areas in the reach from Gatun Lake to Miraflores Lake are higher than the excavation, and are at an average distance of more than 5,000 ft from the center line of the canal. The bottom of the cut through this section is at approximately El. −70 and the average elevation of the top of the spoil areas would be between 400 ft and 500 ft. The distance and necessary gradient to overcome the difference in elevation would necessitate approximately 2 miles of truck haul on an upgrade. Truck haul, therefore, would not be an economical operation for most of the material.

However, a small part of the excavation (that above El. 270) could be hauled economically by trucks because the difference between that elevation and the spoil areas would be sufficiently small to permit removal with an average haul of about 1 mile. In some locations, truck haul could be used economically below El. 270. The exact elevation would be dependent on the location and elevation of the dry spoil areas as well as on the haul distance to the wet spoil area.

The difference in elevations between the bottom of the cut and the disposal areas would require approximately 5 miles of railroad on a 2% grade for the entire distance. Removal by this method, therefore, was not considered feasible or economical.

The large amount of rainfall in this area, combined with the adhesive or sticky quality of the shale which forms a large part of the excavation, and the necessity for breaking the material into small sizes do not favor the use of belt conveyers. In some sections of the canal excavation, this method might be used successfully but it does not appear feasible or economical for the operation as a whole.

The lack of suitable spoil areas within reasonable overland haul distances led to the consideration of a method of disposal in areas in Gatun Lake remote from existing and future channels. Actually, this use of Gatun Lake is not new since the lake has been used for wet disposal of excavated materials since it was first created. The principal new problem which this method of disposal presents, therefore, is that of effecting the transfer of dry excavated materials to suitable haulage equipment and their transportation to the dumping areas. These areas could be made accessible to railroad haul, averaging 15 miles, but trestles would have to be constructed in Gatun Lake. Furthermore, it was found that the material could be hauled more economically in scows. The Gatun Lake spoil areas are readily accessible to bottom dump scows and have capacity for more than 3,000,000,000 cu yd of spoil material below El. 65. Accordingly, a method of dry excavation was developed so that the material could be handled directly into scows to take advantage of the low cost water transportation. The haul distance from point of excavation to point of disposal would vary from 1 mile to 23 miles and would average about 15 miles. The average cost of disposal by truck haul would be approximately twice as much as by scow haul.

Practically all disposal of dry excavation could be handled by the scow haul method by using a system of auxiliary barge channels, which would lie within the limits of the excavation area and would roughly parallel the center line of the new channel for the entire length of the dry excavation (Figs. 98 and 99). The initial auxiliary barge channels would enter the sea-level canal channel from the lake through natural inlets, or in some instances by way of an inlet or a channel excavated to connect with the present canal. In constructing the barge channels, a pilot cut 180 ft wide to El. 90 would be made first along the length of the cut, followed immediately by a channel 110 ft wide excavated to El. 65, giving the channel a depth of 20 ft with the lake surface at El. 85. Material adjacent to both sides of the barge channel would be excavated to El. 90 casting directly, or recasting, into bottom dump scows in the barge

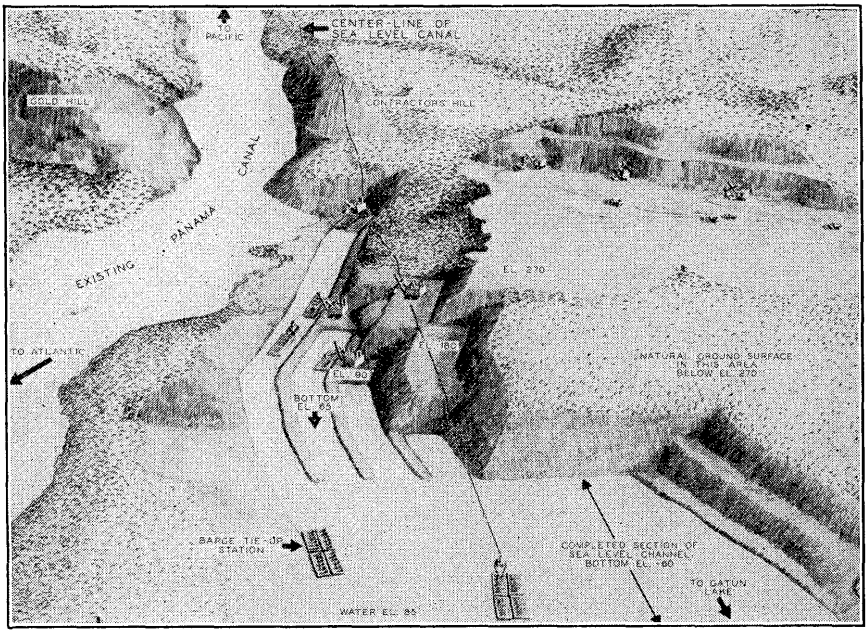

FIG. 98.—EXCAVATION ABOVE EL. 90, APPROACHING CONTRACTORS HILL FROM THE NORTH

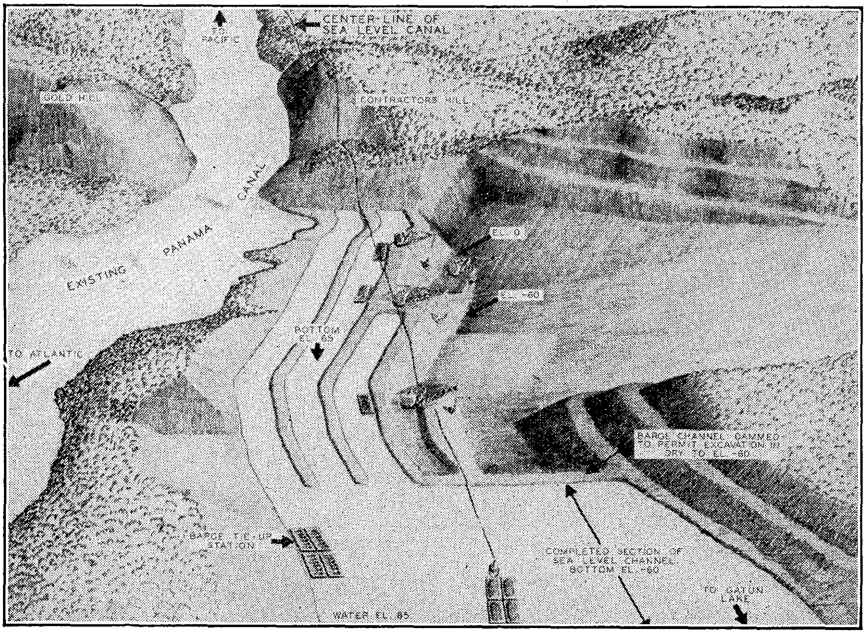

FIG. 99.—EXCAVATION BELOW EL. 90, APPROACHING CONTRACTORS HILL FROM THE NORTH

channel. When a width of 180 ft had been excavated on either side of this channel, another barge channel would be excavated parallel to it, leaving a 70-ft berm between the two channels as shown in Fig. 98. This process would be continued until the excavation to El. 90 had been extended laterally to intersect the design side slopes at this elevation. All the material above El. 90 could be loaded or cast directly into the scows except that for the excavation of the very flat slopes, which would require recasting or trucking of the material to the scows.

For the excavation of material below El. 90 to project depth (Fig. 99), the barge channels would be blocked and dewatered progressively from one side of the channel to the other, each being excavated in turn to project depth. After drilling and blasting, the material in each dewatered channel would be removed by a dragline operating from the 70-ft berm separating the dewatered channel from the adjoining open or wet channel. Material below El. 90 would be excavated in two steps. In the first step, when the material was excavated to El. 0, the draglines would pick up the materials and load directly into the scows. In the second step, below El. 0, the materials would be cast on a bench and then picked up by draglines and loaded into scows.

The scow haul method of disposal facilitates the construction of the broad flood-control dams in Gatun Lake. This method would materially reduce the construction costs of these dams because they could be built to El. 65 with the 2,000-cu-yd bottom dump scows with very little extra haul distance. The dams could then be raised by using smaller barges, which would permit dumping to El. 75. The only important cost for the construction of the dams would be the special handling of a small part of the material from El. 75 to the required height at El. 82.

The scow haul method of disposal of all material excavated below El. 270 was adopted for construction planning purposes because it was the most economical method. The dry spoil areas are favorably located for the disposal of material excavated above El. 270 and the use of 20-cu-yd trucks was adopted for planning purposes. Additional economies may be realized by further study of truck haul for material below El. 270 in some locations.

Excavating Equipment.—The size and type of the hauling unit and the type of material to be excavated are the principal factors in selecting the size of shovel to be used. The capacity of the hauling unit should be equal to one or more times the load handled at one time by the excavator. For the material hauled by truck, studies have indicated that the 5-cu-yd shovel would be the most economical type of equipment for leading 20-cu-yd trucks.

Large excavating equipment, shovels and draglines with capacities greater than 25 cu yd, has proved economical in strip-mining operations where there is no problem of transportation. However, no records are available, involving the use of these large shovels or draglines, where the excavated material had to be transported. The scow haul method, previously described, using auxiliary channels, would lend itself readily to the use of this large equipment. This equipment has a wide operating radius, which permits it to make a cut about 180 ft wide. The large swinging radius of this equipment would facilitate the direct loading of scows reaching the equipment through barge channels.

This equipment could work high faces of 90 ft and the size of the dippers would require less drilling and blasting of the hard material. Also, the rugged construction of the large scows would withstand the loading shock of the material deposited by the large dippers. Furthermore, the large equipment would require less operating personnel for the same volume of material excavated than would smaller excavation equipment.

The lack of mobility of the large equipment would not be a serious disadvantage on this job because the excavation is highly concentrated. Large quantities of material would have to be removed for each position of each piece of equipment, because of the great depths and widths of cut. For the foregoing reasons, the large excavating equipment, having buckets of 25-cu-yd capacity or greater, was adopted for construction planning purposes for the excavation of practically all the material below El. 270.

Drilling and Blasting.—The methods of excavating and of hauling each have a definite bearing on the type and quantity of drilling and blasting required. The character of the material to be blasted determines the selection of the type of drilling equipment. The spacing of holes and the amount of blasting, expressed in terms of pounds of powder per cubic yard, are determined by the size of the hauling, loading, and drilling equipment.

The classification of the material to be drilled and blasted is shown in Table 34. From studies of drilling performed by contractors and The Panama Canal in this area, it was determined that the rotary drill would be the most suitable and economical for soft and medium rocks; and the percussion or churn drill for hard rock.

Excavation with small shovels would be on rather low faces, about 30 ft high, and would require fairly close spacing of drill holes varying from 8 ft by 8 ft for the hard rock, drilled by percussion-type wagon drills, to 18 ft by 18 ft for for the soft rock, which would be drilled with rotary drills.

Most of the dry excavation, 700,000,000 cu yd out of 750,000,000 cu yd, would be handled with the large shovels and draglines, 25 cu yd and more, and hauled in large scows. The digging height of the large shovels is about 90 ft, and therefore depths of drilling of 90 ft would be practicable. The size of the dippers and the fact that the scows could accept pieces as large as those handled by the dipper, would permit 30-ft by 30-ft spacing of the drill holes in soft and medium rocks, materially reducing the cost of drilling. For the hard rock, drilling could be done to the full depth of the 90-ft faces, but coyote or tunnel blasting would be employed to effect savings in drilling and in blasting.

The weight of powder required would vary with the material, size of blast, and local conditions. Past experience in the Canal Zone indicates that the powder requirements will vary between $\frac{1}{4}$ lb per cu yd for the soft rocks to $\frac{3}{4}$ lb per cu yd for the hard rocks, when the drilling and blasting are done in the dry.

WET EXCAVATION

The deep-dredging scheme for conversion of the Panama lock canal to sea level requires the development of special equipment for excavation to the unprecedented depth of 145 ft below the existing level of Gatun Lake. It also calls for subaqueous drilling and blasting to far greater depths than have previously been required in wet excavation. Since these two considerations are controlling factors in setting up the construction procedure for wet excavation, they will be discussed first, followed by a brief discussion of proposed construction procedure.

Dredge Design Contracts.—To obtain the views and advice of the dredging industry as a whole regarding the feasibility of dredging to depths of 145 ft, a preliminary conference was held in Philadelphia in March, 1946, attended by representatives of thirty dredging contractors, dredge designers and builders, the Corps of Engineers, and The Panama Canal. At this conference, problems involved in construction of a sea-level canal were discussed, and it was the consensus that deep dredging was practicable. Subsequently, contracts were made with dredge manufacturers for design of the following types of dredges:

Type	Company	Maximum dredging depth (ft)
Hydraulic	Panama Contractors	145
Bucket ladder	Cuba Manufacturing Company	145
Dipper	Bucyrus-Erie Company	85

Revised specifications of the bucket-ladder dredge show that, with the ladder at a 45° inclination with the horizontal, the maximum dredging depth would be 148 ft.

Panama Contractors, organized especially for the hydraulic dredge contract, is a combination of the Atlantic Gulf and Pacific Company, Standard Dredging Corporation, and Gahagan Construction Company.

The maximum dredging depth of 148 ft specified for the hydraulic and bucket-ladder dredges is that required for excavation to grade, working from the normal Gatun Lake elevation of 85 ft. The maximum dredging depth of 85 ft for the dipper dredge is presently considered to be the greatest depth to which a dredge of this type can economically excavate. It is proposed, therefore, that dipper dredges will excavate rock to an 85-ft depth, followed by deep-digging bucket-ladder dredges, excavating to grade. Bucket-ladder dredges, capable of dredging to a maximum depth of 90 ft, would also be utilized for initial rock excavation. The hydraulic dredge would be used only for the excavation of common material and soft rock, such as the Gatun sandstone. Development designs and cost estimates for the construction of the three types of dredges have been completed.

Hydraulic Dredge.—In present hydraulic dredge design, the main pump of the dredge is located near the water surface. Atmospheric pressure, in forcing the dredged materials to the pump from depths of 40 ft or 50 ft, is sufficient to supply ample velocity head, overcome friction losses, and support a column of mixture having a density substantially greater than that of water. As deeper dredging is undertaken, the entrance loss and velocity head remain the same, but the friction head and the pressure necessary to support the columns of mixture increase. Various schemes to bring a richer mixture to the dredge pump were considered, such as utilizing nozzles in the suction, depressing the elevation of the main pump, placing a propeller-type pump in the suction, or placing a booster pump on the dredge ladder below the water surface. It was

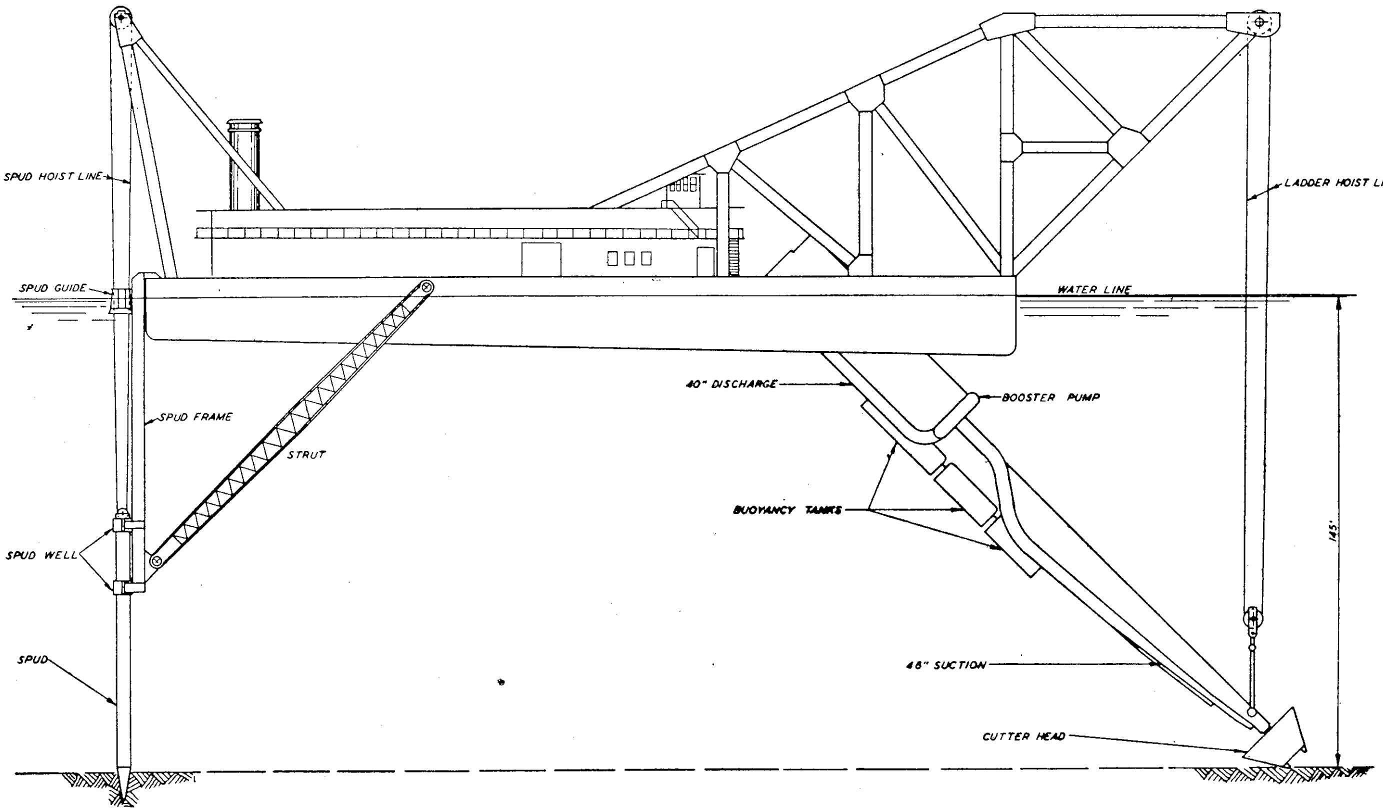

Fig. 100.—Proposed 40-In. Hydraulic Dredge

decided that the last scheme was the most practicable of those considered. Fig. 100 shows a general arrangement of the ladder and booster pump.

Two basic mechanical problems that required consideration were: First, the design of a dredge ladder approximately 200 ft long and capable of supporting the booster pump and attendant machinery; and, second, the design of spuds and a spud frame to hold the dredge in place. Both designs were developed sufficiently to indicate that they are entirely practical.

The deep-dredging ladder is provided with buoyancy tanks (Fig. 100) for assistance in hoisting and for better stress distribution. The booster pump is enclosed in a cofferdam to permit removal of obstructions from the booster pump when the ladder is not completely raised.

For holding the dredge in place for deep dredging, a frame extending 90 ft below the water surface is used to hold the spuds. This frame, which is braced to the hull, would be replaced by shorter frames when dredging to more conventional depths. The frame would consist of two main vertical members of sufficient size and section to serve as wells for the spuds which would extend below the wells to a depth of some 160 ft. Fig. 100 shows a schematic layout of this arrangement. General features of the hydraulic dredge are as follows:

Feature	Description
Suction	46-in. diameter
Discharge	40-in. diameter
Hull	265 ft long, 60-ft beam, and 20 ft deep
Power: Main pump	Steam turbine, reduction gears, 8,000 hp normal, and 10,000 hp maximum
Auxiliaries	Electrically driven from turbogenerators
Cutter motor	Electrically driven, 2,000 hp to 2,500 hp
Booster	Electrically driven from shore, 8,000 hp

Bucket-Ladder Dredge.—The bucket-ladder dredge has been used quite extensively in Europe for many types of subaqueous excavation and is generally recognized as a European development. It was the most successful type of dredge used by the French at the Panama Canal, and under the American regime several of the old French bucket-ladder dredges were rebuilt and placed in service, one of these remaining in commission until 1920. Use of the bucket-ladder dredge in the Western Hemisphere, the Dutch East Indies, the Federated Malay States, and Russia has been confined principally to placer mining for gold, platinum, and tin. The bucket-ladder dredge would be used on the Panama Canal project for excavation of blasted rock, work which has been performed successfully by such equipment but which is more unusual than the excavation of gravel and conglomerate.

The bucket-ladder dredge has certain distinct advantages over other types. For example, its operation involves a minimum of lost motion as compared with the dipper dredge; it does not require large quantities of water, as does the hydraulic dredge; it requires relatively little power; and it can dredge to great depths. One bucket-ladder dredge in the California gold fields is excavating to a depth of 124 ft, and 200-ft dredging depths have been predicted for the near future by leaders in the placer dredging industry.

Bucket-ladder dredges of two limiting digging depths are under consideration—one for dredging to a maximum depth of 90 ft and the other for dredging to a maximum depth of 145 ft. The general scheme of operation would be to utilize the 90-ft dredge for excavation to 90-ft depths, followed by deep-digging dredges for excavation between 90-ft and 145-ft depths. For excavation to 90 ft below water level, a dredge hull of shorter length could be utilized than for 145-ft digging; also the length of the digging ladder could be reduced from 239 ft to 163 ft. The original and operating costs of the 90-ft dredge would obviously be less than those of the 145-ft dredge. However, machinery for dredges of both sizes would be identical to permit interchangeability of parts.

The major item of cost involved in rock excavation in the wet is that of drilling and blasting, and, in general, unit costs decrease rapidly as the spacing between drill holes is increased. In the bucket-ladder dredge design, therefore, it was desired to provide the largest practicable buckets to permit maximum spacing of drill holes. Consideration was given to bucket sizes of 6 cu yd, 5 cu yd, 4 cu yd, 3 cu yd, and 2 cu yd. Representatives of the contractors and The Panama Canal agreed that the 2-cu-yd bucket is the maximum size that should be adopted, because of difficulties in casting very large buckets and because of the tremendous weight of the digging ladder and the bucket chain.

The 2-cu-yd bucket has three times the capacity of the largest buckets now in use in placer dredging. A few European dredges have been equipped with buckets having capacities up to 2 cu yd, but the working conditions that they encountered were not severe, the maximum dredging depth being only about 50 ft and the material excavated being sand, silt, shell, and clay. The bucket-ladder dredge *Corozal*, formerly owned by The Panama Canal, was equipped with 2-cu-yd buckets for soft digging and $1\frac{1}{4}$-cu-yd buckets for rock excavation.

Spoil disposal would be accomplished by scows, loaded by conveyers on the starboard and port sides of the dredge. Consideration was also given to a stern conveyer system composed of the necessary conveyer unit for final disposal of spoil ashore. However, detailed study indicated that the latter method of disposal is not desirable because of the very long conveyer lines required and because of difficulties in designing the spud arrangement. By elimination of the stern conveyer system, spud design is simplified. Fig. 101 shows an elevation of the proposed bucket-ladder dredge, including the proposed spud arrangement.

General features of the bucket-ladder dredge design for 145-ft dredging as developed under contract are as follows:

Description	Quantity
Dimensions of Hull, in Feet—	
Length	371
Beam	100
Depth	13
Bucket capacity, in cubic yards	2
Power; number of 1,600-hp, deisel electric engines with identical direct-connected generating equipment	4
Main drive, in horsepower	2,500
Total connected load, in horsepower	5,294
Length of digging ladder, in feet	239

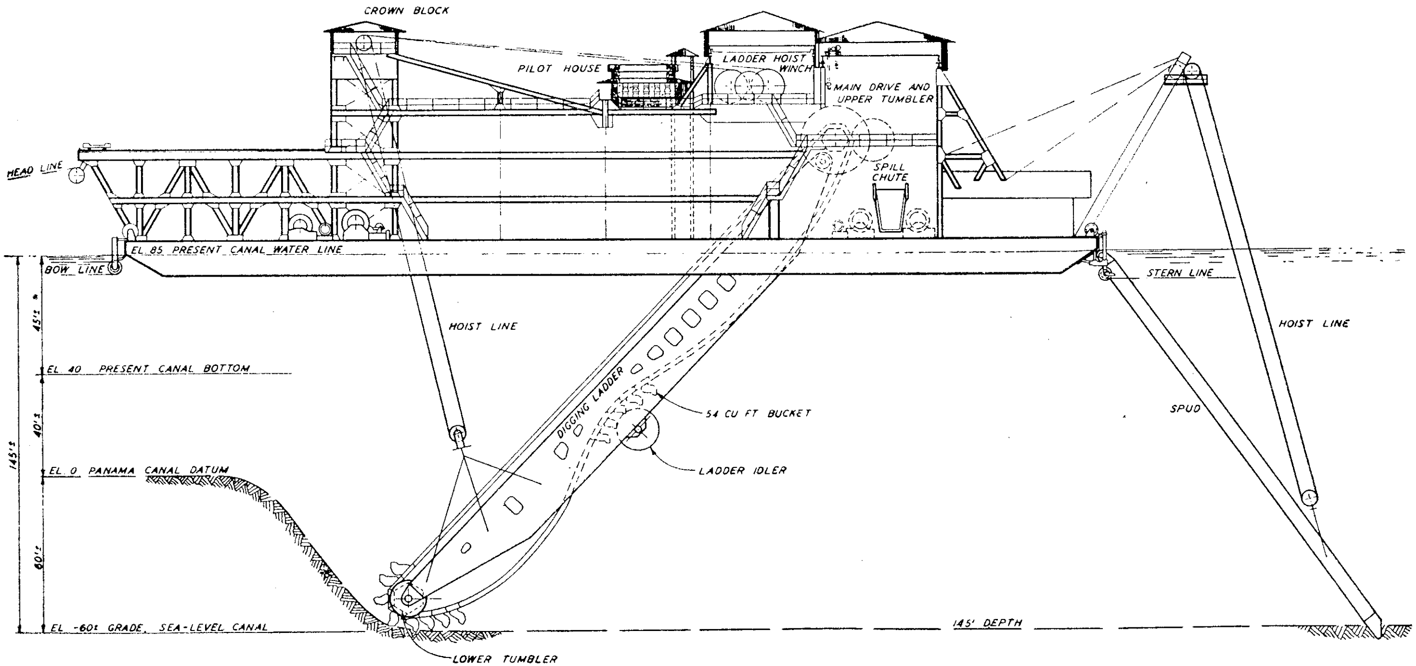

Fig. 101.—Proposed 54-Cu-Ft Bucket-Ladder Dredge

Dipper Dredge.—The dipper dredge is a proved tool for rock excavation, its principal limiting factor being that of digging depth, since the time required for the digging cycle increases rapidly as depths increase. Although, under the deep-dredging plan, the dipper dredge contemplated could not excavate to final grade from the existing level of Gatun Lake, its maximum dredging depth of 85 ft is considerably in excess of that of any dipper dredges previously developed. Furthermore, large quantities of hard rock are located above the 85-ft depth in Gatun Lake (that is, above El. 0) within the economical digging range of the dipper dredge. After canal construction, the dipper dredge could be used effectively for maintenance.

The dipper dredge design incorporates a balanced hoist and a walking stern spud, which are shown diagrammatically in Fig. 102.

The balanced hoist consists of a movable counterweight connected to the main hoisting machinery by a double three-part hoist. The counterweight is lowered as the dipper is raised, or vice versa, effecting an approximate balance for the dead weight of the dipper and handle, so that practically the entire hoist motor power is available for digging. The movable counterweight is located at the stern.

The stern spud would be mounted as a trailing or walking spud in a well at the stern of the hull. For the purpose of controlling the angular movement and applying power to walk the spud or push the dredge ahead, a powerful hydraulic ram on the main deck would be pin-connected to a heavy frame surrounding the spud.

The dredge would be electrically operated from a 4,000-v, three-phase, 60-cycle alternating current generated aboard by four identical diesel engine generating sets. The combination of any three sets would provide sufficient power for full-speed operation of the dredge, and the fourth would be a stand-by unit. General features of the dipper dredge design are:

Description	Quantity
Length of Hull, in Feet—	
Length	200
Beam	85
Depth	14
Dipper capacity, in cubic yards	20 to 30
Power; diesel electric power from 1,000-hp engines (adaptable for shore power)	....
Maximum digging depth, in feet	85
Bail pull, in pounds (stalling)	500,000

Deep Drilling and Blasting for Wet Excavation.—Preparation of rock for excavation under the deep-dredging plan requires subaqueous drilling and blasting to depths far in excess of those ever before encountered in canal excavation. Therefore, it was considered necessary to determine the effects of these depths, as compared to conventional depths, on (1) drilling and blasting operations, including maneuvering of equipment and rate of drilling; (2) requirements of explosives; and (3) dimensions of the blasted rock.

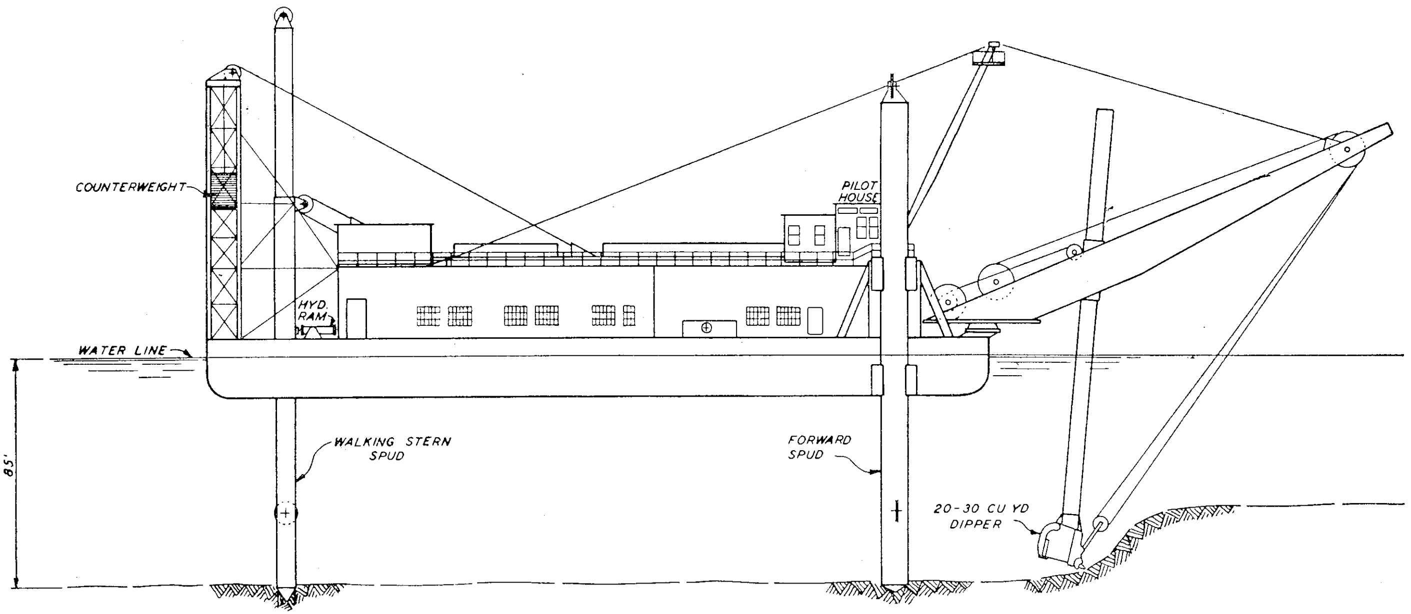

Fig. 102.—Proposed 20-Cu-Yd to 30-Cu-Yd Dipper Dredge

To obtain the desired information, a deep-drilling and blasting test was conducted in which rock was drilled, blasted, and excavated in three lifts of approximately 25 ft each from 50 ft to 125 ft below water surface. An accurate record was kept of all drilling, loading, and blasting operations to determine drill performance, the difficulty and time required for maneuvering equipment, setting casing, and loading. Fragmentation of the excavated material was observed to ascertain the effect of hydrostatic head on breakage. The same quantity of explosive per cubic yard was used for each lift.

All drilling was performed by rotary drill, mounted on a barge with necessary winches for maneuvering. Excavation was accomplished by derrick barge equipped with a 3-cu-yd clamshell bucket. From the test, it was concluded that:

(1) Drilling operations with a properly designed drill boat are not appreciably affected by the depths of water that would be encountered in the deep-dredging plan.

(2) The effect of increased depth of water (within the limits of this test) is negligible.

(3) Dimensions of blasted rock obtained in all three lifts did not differ appreciably. The same unit explosive loading and hole spacing were used for each lift.

The rotary drill was selected because it could be utilized in the test without extensive and costly alterations, whereas other types of equipment would have required major reconversion with resulting delay of the test. Furthermore, sufficient data regarding drilling rates at conventional depths with the percussion and rotary types of drills were available to permit reasonably accurate conversion of deep-drilling results with one type of drill to those that might be expected with the other.

In the preparation of estimates for drilling, blasting, and dredging, rock has been divided into three classifications—soft, medium, and hard—on the basis of extensive core borings. Experience on the Panama Canal indicates that, for soft and medium rocks, subaqueous drilling rates are higher and costs are lower, using the rotary type of drill. For hard rock, drilling rates of the rotary and percussion drills are almost equal, or slightly favor the rotary type, but bit costs are higher for the rotary drill. The net result is that in hard rock the drilling cost per foot for the rotary drill may be somewhat higher than that for the percussion type. This latter factor, however, is not important because hard rock comprises only 5% of the total subaqueous drilling and blasting in the project, or 4% of the deep subaqueous drilling and blasting.

Wet Excavation Procedure.—Excavation would be performed in the dry throughout the Gatun Lake section to El. 90 (5 ft above normal lake level) and throughout the Miraflores Lake section to El. 60 (6 ft above normal Miraflores Lake level). Excavation below El. 90 in Gatun Lake and below El. 60 in Miraflores Lake, except for certain sections adaptable to dry excavation behind plugs, would be performed by dredges. Dredging conditions in both the Atlantic and Pacific sea approaches are suitable for the use of existing hydraulic and dipper dredges.

The deep-dredging plan would require that about 60% of the total dredging be performed to depths of 85 ft or less; and about 45% of the total dredging, to depths of 65 ft or less. Therefore, a number of present-day dredges that can excavate to a depth of 65 ft with minor or no alterations and some dredges capable of excavating to 85-ft depths without extensive modification could be used on the project.

Plans contemplate the use of conventional hydraulic dredges for the initial excavation of common material to a depth of 65 ft, or possibly to a depth of 85 ft. Common material from the bottom of the initial excavation to a depth of 145 ft would be removed by the 40-in. hydraulic dredges previously described. The number of passes or lifts that would be required in excavation to final grade would vary with materials and their ability to flow to the dredge suction without objectionable caving of the bank, but usually the excavation would be performed in about 30-ft lifts. Hydraulic dredge spoil would be pumped to the nearest available disposal area, and for most of the material in Gatun Lake only a floating pipe line would be required. Pipe-line lengths would vary from 3,000 ft to 12,000 ft and, when in excess of about 5,000 ft, shore-powered boosters would be introduced into the line. Pipe lines would be laid to disposal areas on both sides of the canal so that there would be no interference with traffic. For example, a dredge working east of the center line would discharge to the east, and vice versa. Under certain conditions, where it might be necessary to pump across the channel, submerged pipe lines would be used.

Following the removal of common material or overburden by hydraulic dredge, rock would be drilled and blasted for subsequent removal by dipper and bucket-ladder dredges. Most, if not all, of the drilling and blasting would be performed by rotary drill boat.

Blastholes would be overdrilled the equivalent of one half of the hole spacing, which would vary from 10-ft centers in hard rock to 14-ft centers in soft rock. It is anticipated that lifts of about 30 ft each would usually be blasted, but this estimate would vary considerably, depending on local conditions and type of excavating machinery utilized. Future drilling and blasting tests may indicate the desirability of drilling and blasting to grade in one operation. Blastholes would be drilled approximately 6 in. in diameter, and for the larger blasts as much as 20,000 lb of dynamite would be fired in one shot.

Following drilling and blasting operations, rock would be excavated by dipper and bucket-ladder dredges, loaded into 2,000-cu-yd dump scows and towed to the dumping grounds by 1,500-hp tugs. Spoil from the Gatun Lake area would be utilized in the construction of the flood-control dams across Gatun Lake, whereas that from Miraflores Lake and from the Atlantic and Pacific approaches would be deposited at sea.

Summary

The methods presented in this paper are feasible and show a decided saving over other methods studied. The estimate showed that approximately 15% could be saved on the cost of the dry excavation alone, whereas the use of the large dry excavation equipment would reduce the required personnel by 9,000 workers, or by approximately 30%. This saving would be reflected in the lowered cost of housing, utilities, services, and mobilization, making an over-all saving of between 10% and 12% for the project. In addition to the cost saving, there would be a saving in the time required for construction. Any reasonable schedule to perform the required excavation by stage dredging would cover a 15-year period, whereas the adopted plan could be scheduled for a 10-year construction period.