GEOTECHNICAL INVESTIGATION
ORLEANS LEVEE DISTRICT
LONDON AVENUE OUTFALL CANAL
OLB PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

VOLUME I

FOR
THE BOARD OF LEVEE COMMISSIONERS OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
ENGINEERS, PLANNERS & ENVIRONMENTAL SCIENTISTS
NEW ORLEANS, LOUISIANA

By
Eustis Engineering Company
Metairie, Louisiana

4 March 1986

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4 March 1986

Burk and Associates, Inc.
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Attention Mr. Michael Jackson

Gentlemen:

Draft of Report
Geotechnical Investigation
Orleans Levee District
London Avenue Outfall Canal
OLB Project No. 2049-0269
New Orleans, Louisiana

Transmitted is a draft of our engineering report for the subject project.

Following your review and comments, we will issue the final report.

Yours very truly,

EUSTIS ENGINEERING COMPANY

By Lloyd A. Held, Jr.

LAH:bh

Enclosures
(Report in Two Volumes)

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GEOTECHNICAL INVESTIGATION

ORLEANS LEVEE DISTRICT

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INTRODUCTION

1. This report contains the results of a geotechnical investigation performed for the proposed London Avenue Outfall Canal between Lake Pontchartrain and the Sewerage and Water Board Pumping Station No. 3 in New Orleans, Louisiana. Authorization to proceed with the investigation was received on 24 September 1985 from Mr. Earl J. Magner, Jr., Chief Engineer of The Board of Levee Commissioners of the Orleans Levee District. Burk and Associates, Inc., New Orleans, Louisiana, are the consulting engineers for the project.

2. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of The Board of Levee Commissioners of the Orleans Levee District and their representatives for specific application to the subject site. In the event that any changes in the nature, design or location of the proposed structures and canal are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

3. The analyses and recommendations contained in this report are based in part on data obtained from the soil borings. The nature and extent of variations in the subsoil conditions may not become evident until construction. If variations then

appear, it will be necessary to re-evaluate the recommendations contained in this report.

SCOPE

4. The scope of the investigation included the drilling of soil borings to determine subsoil conditions and stratification and to obtain samples of the various strata encountered. In situ Standard Penetration Tests and soil mechanics laboratory tests performed on undisturbed samples obtained from the borings were used to evaluate the physical properties of the subsoils. Engineering analyses, based on the borings and laboratory tests results, were made to determine allowable pile load capacities, slope stability analyses, cantilever sheetpile analyses, seepage analyses, and estimates of settlement. Additionally, the scope included the installation of piezometers to evaluate underseepage potential in the area.

SOIL BORINGS

5. A total of ninety-eight (98) soil test borings were drilled during the period 3 October - 17 December 1985 at the approximate locations shown on Figure 1 and tabulated in Appendix A. Sixty-nine of the borings were drilled at the centerline of the existing levee or at the existing levee toe. Sixty-three of these borings were 3-in. diameter borings drilled to various depths. Forty-eight of these borings were drilled to a depth of 50 feet, three to a depth of 65 feet, three to a depth of 70 feet, three to a depth of 80 feet, and nine to a depth of 100 feet below the existing ground surface. Borings 45, 56 and 65 were 5-in. diameter borings drilled to a depth of 50 feet below the existing ground surface. The remaining twenty-nine borings were drilled in the canal to a depth of approximately 10 feet below the canal bottom or until a sand stratum was encountered. Originally, thirty-two canal borings had been proposed. Proposed Borings 70, 71 and 72 could not be drilled because the canal was

lined with a concrete slab in the vicinity of these borings. The results of the soil borings are shown in both tabular and graphical form on the individual boring logs in Appendix A.

6. Two (2) additional continuous 5-in. diameter borings were drilled during the period 23-25 October 1985. Cohesive and semi-cohesive samples were retained in thinwalled tubes and transported to the U.S. Army Corps of Engineers, New Orleans District. Non-cohesive samples were obtained during Standard Penetration Tests, retained in glass jars and transported to the New Orleans District. Driller's logs for these borings were also forwarded to the New Orleans District.

7. The borings drilled on land were made with a truck mounted rotary type drill rig. Samples of cohesive or semi-cohesive soils were obtained using either a 3-in. or 5-in. diameter Shelby tube sampling barrel. Samples were generally obtained at close intervals or at changes in soil strata. Continuous samples were obtained in the three 5-in. diameter borings and in three of the 3-in. diameter borings drilled to a depth of 50 feet below the existing ground surface. Samples were extruded from the sampling barrel, inspected and visually classified by Eustis Engineering Company's soil technician. Representative samples were placed in moisture proof containers and sealed for preservation.

8. Cohesionless soils, when encountered, were sampled during the performance of in situ Standard Penetration Tests. These tests consist of driving a 2-in. diameter sampler one foot into the soil, after it is first seated 6 inches, using blows of a 140-lb weight dropped 30 inches. The number of blows required to drive the sampler one foot is recorded and is indicative of the relative density or approximate consistency of the subsoils encountered. The results of the Standard Penetration Tests are recorded on the boring logs.

9. The borings drilled in the canal were sampled with a 2-in. diameter piston sampler. Samples were sealed for preservation prior to laboratory testing.

PIEZOMETER INSTALLATIONS

10. Piezometers have been installed in the near surface sand strata at depths and locations indicated on the tabulation below.

Piezometer	Location	Depth Below Existing Ground Surface In Feet
P-1	Station 149+00, West Levee Toe	15
P-2	Station 149+00, 100 Feet West of West Levee Toe	15
P-3	Station 101+00, West Levee Toe	20
P-4	Station 101+00, 75 Feet West of West Levee Toe	20
P-5	Station 39+80, West Levee Toe	20
P-6	Station 39+80, 70 Feet West of West Levee Toe	20

Details of a typical piezometer installation are shown on Figure 2. Piezometers should be sounded on a periodic basis in order to establish any correlation between stages in Lake Pontchartrain and piezometric head in the near surface sand strata.

LABORATORY TESTS

11. Soil mechanics laboratory tests consisting of natural water content, unit weight and either unconfined compression shear or unconsolidated undrained triaxial compression shear were performed on undisturbed samples obtained from the borings. Atterberg liquid and plastic limits determinations were performed on representative samples of cohesive soils obtained from the borings. The results of the laboratory tests are shown in tabular form in Appendix B.

12. Additional soil mechanics laboratory tests performed by the U.S. Army Corps of Engineers were performed for previous investigations within the project areas. Tests which are considered pertinent to this investigation are shown in Appendix C.

DESCRIPTION OF SUBSOIL CONDITIONS

Reach I (Station 0+00 to Station 21+00)

13. Reach I extends from Sewerage and Water Board Pump Station No. 3 proceeding north towards Lake Pontchartrain and terminates at the northern end of the concrete lined portion of the London Avenue Canal. Based on the furnished information, the existing levee crown varies in elevation from approximately el 4 to 7 msl (Mean Sea Level). The ground elevation at the toe of the levee varies from approximately zero to 4.

14. Reference to the individual logs of borings contained in this report for this reach shows that the levee fill materials are generally composed of soft to stiff gray and tan clay and silty clay with roots, sandy silt, clayey silt and silt lenses and layers and pockets extending to elevations varying between -2 and -4. At Boring 1, fill materials are encountered to approximate el -15 and are probably associated with the construction of Pumping Station No. 3 near this boring. Beneath the levee fill materials and continuing to the Pleistocene contact encountered at elevations varying between -62 and -70 are alternating strata of soft to medium stiff gray clay, silty clay and sandy clay and loose to medium dense clayey silt, clayey sand, sandy silt and sand.

15. Pleistocene deposits are characterized by reworked Pleistocene material encountered at Borings 1, 2, 36 and 37 from approximately el -58 to -67. These are generally a stratum of medium stiff gray, tan or brown clay with silt pockets and lenses, sand layers and shell fragments. Beneath these deposits and continuing to the termination of the borings are strata of

stiff to very stiff greenish-gray, gray or tan clay and silty clay with silt lenses, layers and pockets. These strata appear to be fissured in areas.

Reach II (Station 21+00 to Station 37+00)

16. Reference to the furnished cross-sections indicates that the elevation of the crown on the levee varies between el 5 and 6. The existing ground surface elevation adjacent to the levees varies between el 3 and -3.

17. Individual logs of borings contained in this reach indicates that the levee fill materials are generally composed of soft to stiff tan, gray or brown clay with roots and clayey silt and silt lenses and pockets. These materials are encountered from the levee crown to elevations ranging between 1 and -1. These deposits are generally underlain by a stratum of medium stiff to stiff gray and tan clay with roots, organic matter and clayey silt and humus pockets to approximate el -18. At that elevation and continuing to approximate el -40 are strata of soft to medium stiff gray clay with roots, organic matter, sand, silty sand and silt lenses and layers. At approximate el -40 and extending to the termination of all of the borings within this reach are strata of very loose to medium dense gray clayey sand and sand with clay layers and shell fragments. Borings within this reach were terminated in the Recent deposits and the Pleistocene contact was not encountered.

Reach III (Station 37+00 to Station 120+00)

18. Reference to the furnished ground surface elevations indicates that the elevation of the crown of the levee varies from approximately el 6 to el 3. The existing ground surface elevation adjacent to the levees varies between approximate el 1 and -6.

19. Individual logs of the borings contained in this reach indicates that the levee fill materials are generally com-

posed of medium stiff to very stiff gray, brown or tan clay or silty clay with wood, roots, organic matter, gravel and clayey sand and sand lenses and pockets. Borings 56 and 57 indicate that the levee fill materials are comprised of medium compact and medium dense brown or gray clayey silt and clayey sand. Levee fill materials extend from the levee crown elevation to elevations varying between -2 and -5. Levee fill materials are generally underlain by strata of soft to medium stiff gray clay with wood, roots, organic matter and clayey silt and silt lenses, pockets and layers. These deposits are not encountered at Boring 57 and generally extend to elevations varying between -10 and -16 at the remaining borings. Anomalous to this general stratigraphy is a clayey silt stratum that extends from approximate el -16 to el -23 at Boring 20. Of particular note are several areas of buried wood within these deposits. Beneath these deposits and extending to elevations ranging between -32 and -45 are strata of loose to dense gray sand with shell fragments and clay layers. These strata are very loose and very dense in areas and are underlain at some locations by strata of very loose to medium dense gray clayey sand and silty sand or soft to medium stiff gray sandy clay with shell fragments. These are in turn underlain by medium stiff to stiff gray clay with sand and silty sand lenses and pockets and shell fragments that are encountered to the Pleistocene contact at elevations ranging between -54 and -60. An organic clay stratum is encountered at Boring 57 between el -53 and the Pleistocene contact at approximate el -58.

20. Surficial Pleistocene deposits within this reach are generally stiff to very stiff greenish-gray or tan clay and silty clay with clayey silt and sand lenses, pockets and layers extending to elevations varying between -65 and -72. Beneath these deposits, borings within this reach encounter strata of medium dense to dense tan or gray sand or loose to compact gray or tan clayey silt with clay layers and shell fragments. These deposits are underlain to the termination of the deep borings in this reach by strata of medium stiff to stiff gray clay with silt and sandy silt lenses.

Reach IV (Station 120+00 to Station 127+00)

21. Reference to the furnished ground surface elevations indicates that the elevation of the crown of the levee varies from approximate el 10 on the west side to approximate el 5 on the east side of London Avenue Canal. The existing ground surface elevation is approximately at el -5 at the east side levee.

22. Individual logs of the borings contained in this reach indicates that the levee fill materials are composed of medium stiff to stiff gray, brown or tan clay and silty clay, medium dense to dense gray or tan silty sand or very compact brown and gray clayey silt. Levee fill deposits extend from the levee crown to elevations ranging between -5 and -10. These materials are underlain by alternating strata of gray clay and organic clay to depths ranging between -15 and -20. At Boring 29, a loose humus deposit is encountered, between el -8 and el -15. Interspersed within these deposits are several areas of wood. Beneath these deposits and continuing to elevations varying between el -46 and el -50 are strata of loose to dense gray sand with clay layers and shell fragments. Beneath the sand strata and continuing to the Pleistocene contact generally encountered between el -55 and el -60 is a stratum of medium stiff gray clay with clayey sand pockets and shell fragments. At Boring 61, a loose clayey sand stratum intersperses the sand and clay deposits between el -35 and el -42.

23. Surficial Pleistocene deposits are generally characterized by stiff greenish-gray and tan clay with sand and silt pockets extending to approximate el -67. At that elevation and continuing to approximate el -82 are encountered strata of medium compact to compact or very dense tan or gray sandy silt or silty sand. These are generally underlain to the termination of all the deep borings in this reach by a stratum of stiff gray or tan clay with silt pockets.

Reach V (Station 127+00 To Lake Pontchartrain)

24. Reference to the furnished ground surface elevations indicates that the elevation of the crown of the levee varies between el 11 and 9. Existing ground surface elevations adjacent to the levees vary between approximate el 5 and el 0.0.

25. The individual logs of the borings contained in this reach indicate that fill material does extend from the crown of the levee to elevations ranging between el -8 and -10. These fill materials are extremely variable and are comprised of soft to medium stiff, loose to medium compact, or loose to medium dense gray brown or tan clay, clayey sand, sand and clayey silt. These deposits contain many roots and shell fragments. Fill materials are generally underlain by strata of very loose to medium dense gray sand, silty sand and clayey sand interspersed with soft gray clay layers and very loose gray clayey silt layers. These deposits continue to elevations varying between el -23 and -40. These strata, in turn, are generally underlain by strata of loose to medium dense gray sand with clay layers and shell fragments. The majority of the borings taken within this reach terminates in these sand deposits. At Borings 29 and 63, these sand deposits extend to approximate el -48 and are underlain by a stratum of medium stiff gray clay. This stratum extends to the Pleistocene contact which is encountered between el -55 and el -60.

26. As indicated by Boring 63, the surficial Pleistocene deposits at the south end of Reach V are characterized by a stratum of stiff greenish-gray and tan clay with sand and clayey silt lenses and pockets that extends to approximate el -68. From el -68 extending to approximate el -78 are strata of loose to medium dense gray sand and silty sand underlain to the termination of the boring by a stratum of medium stiff gray clay.

Canal Borings

27. Reference to the furnished cross-sections indicates that the canal thalweg varies in elevation from approximately el -5 at the south end of the proposed project near Pumping Station No. 3 to elevations ranging between -10 and -13 in the reaches extending to Leon C. Simon Boulevard (approximately Station 127+00). From Leon C. Simon Boulevard to Lake Pontchartrain, the elevations of the canal bottom vary considerably due to siltation from Lake Pontchartrain. Cross-sections indicate that the elevations range from approximately el -2 to -13.

28. Attempts to make canal borings between Station 0+00 and Station 27+00 indicate that the canal is concrete lined within this reach. From Boring 73 (Station 19+60) to Boring 76 (Station 34+60), deposits of soft to medium stiff gray clay which underlie 1 to 2 feet of extremely soft black muck are generally encountered. From Boring 77 (Station 39+60) to Boring 89 (Station 99+75), loose to medium dense gray sand strata are generally encountered near the canal bottom. These strata underlie a variable cover of 1 to 5 feet of extremely soft to very soft gray clay and organic clay materials. The sand stratum is generally exposed at Borings 79 and 80 and underlies approximately 3.5 feet of loose gray clayey sand at Boring 82. From Boring 90 (Station 104+75), canal borings generally encounter strata of very loose to loose or extremely soft to soft gray clayey sand, silty sand and sand and clay to Lake Pontchartrain.

Ground Water

29. Piezometers installed for the purpose of evaluating underseepage potential in the area were placed in near surface sand strata and are indicative of the depth to ground water. Initial readings subsequent to development of these piezometers are tabulated below.

Piezometer	Depth to Ground Water Below Existing Ground Surface In Feet
P-1	6.2
P-2	5.9
P-3	8.3
P-4	6.3

30. The depth to ground water is sensitive to climatic variation and will be affected by the level of London Avenue Canal. If important to construction, the depth to ground water should be determined by those responsible for construction immediately prior to beginning work.

FOUNDATION ANALYSIS

Furnished Information

31. Survey Data. Plotted survey cross-sections and survey notes relative to the project baselines and profile elevations at the bridge locations have been provided by Burk and Associates, Inc. to Eustis Engineering Company. Elevations indicated on the plotted cross-sections and all other elevations indicated in this report refer to Mean Sea Level (msl).

32. Proposed Flood Protection. Flood protection proposed adjacent to the London Avenue Canal will be provided by either earthen levee or floodwall. Floodwalls will be either a concrete capped cantilevered sheetpile I-wall or a pile supported concrete structure. Preliminary studies have indicated a pile supported concrete floodwall will be required in some areas of Reach I (Station 0+00 to Station 21+00). It is anticipated that flood protection in other areas can be provided by either a sheetpile floodwall or earthen levee. Gated flood structures will be provided at the railroad crossing located near Sewerage and Water Board Pumping Station No. 3 at the south end of the project and at the existing Benefit Street bridge locations. The

remaining bridge locations will provide flood protection at their approximate existing grade by modifying the bridge structure rather than constructing flood gates.

33. The earthen levee will be constructed to grade with minimum 1 vertical on 3 horizontal side slopes and a 10-ft crown. Floodwalls are assumed to be constructed with the existing flood protection in place such that a minimum 3-ft clear distance is maintained between the existing and the proposed flood protection. This 3-ft clear distance is required for form work.

34. Design Conditions. Proposed levee and floodwall flood protection is to be designed for a net grade of 2 feet above the predicted static water level (SWL). At the north end of the project, it is required that net grade be increased due to wave surges expected from Lake Pontchartrain. The static water level and net grades required for the project are tabulated below.

Location	SWL In Feet - MSL	Net Grade In Feet - MSL
Sta. 0+00 to 120+00	11.9	13.9
Sta. 120+00 to 127+00	11.6	13.6
Sta. 127+00 to 144+50 (Eastside) Sta. 127+00 to 142+50 (Westside)	11.5	13.5
Sta. 144+50 to 147+50 (Eastside) Sta. 142+50 to 145+00 (Westside)	11.5	13.5 to 17.5*
Sta. 147+50 to Lake (Eastside) Sta. 142+50 to Lake (Westside)	11.5	17.5**

* Transition.
** Required for wave run-up.

35. Design Criteria. It is required that earthen levees and levee/cantilever sheetpile floodwalls be designed for a factor of safety equal to 1.3 when evaluated by the LMVD Method of Planes stability analyses. The floodside water level assumed in these stability analyses is to be the SWL.

36. Cantilever sheetpile analyses are to be evaluated with a factor of safety of 1.5 factored into the soil shear strength parameters. It is required that the cantilever sheet-pile analyses assume the floodside water elevation at the top of the floodwall.

37. Gated flood structures and pile supported concrete floodwalls must be evaluated by a "Deep Seated Stability Analysis." This method factors a 1.3 factor of safety into the soil shear strength parameters and sums horizontal forces in order to evaluate any unbalanced load on the pile supported structure. Details of this method have been supplied to Burk and Associates, Inc. and in turn to Eustis Engineering Company by the U.S. Corps of Engineers.

38. Steel sheetpile cutoff must be used beneath the gated flood structures and pile supported concrete floodwalls in order to provide protection against possible piping during hurricane conditions. The sheetpile penetration should be based on appropriate seepage analyses. Possible piping should also be evaluated at the cantilever sheetpile wall sections.

39. Supplemental Soil Data. Eustis Engineering Company has been supplied the results of laboratory test data performed on samples obtained from borings taken by the U.S. Army Corps of Engineers, New Orleans District, New Orleans, Louisiana.

40. Preliminary stability analyses have indicated right-of-way requirements preclude the use of earthen levees to provide flood protection in Reaches I, II, III and IV (East). Results of these analyses are shown on Figure 3.

Levee Analyses

41. Settlement Analyses. Eustis Engineering Company has estimated settlement of the levee sections required in Reaches IV (West) and V. Results of our analyses indicate that settlements on the order of 1.5 to 2.5 feet may be anticipated

due to the variable nature of the foundation deposits. Based on the drainage characteristics of the compressible foundation strata, Eustis Engineering Company estimates that a considerable portion of this settlement will occur during construction and that the remaining settlement will occur over a relatively long period of time subsequent to completion of the levee section. Eustis Engineering Company therefore recommends that the net levee section be constructed to a gross grade of one foot above the net levee elevation, and, if necessary, net grade be restored during maintenance. Overbuild of the net levee section in excess of one foot would result in larger setback distances from the existing canal bank and the inclusion of stability berms or larger stability berms than the ones indicated in this report for landside stability. We should note that placement of fill adjacent to existing structures may result in possible settlement of these structures. Eustis Engineering Company requests that when the final alignment is determined, we be furnished information regarding the proximity of existing structures to the fill in order that we can determine if such settlements will occur.

42. Stability Analyses. Stability analyses for Reach IV (West) and Reach V are shown on Figure 4. These analyses indicate a minimum factor of safety of 1.3 when evaluated by the LMVD Method of Planes analyses. Stability analyses are based on composites of existing cross-sections overlaid about the centerline of the existing flood protection. An average shear strength trend within the clay strata was developed based on the results of laboratory tests conducted by Eustis Engineering Company. Stratigraphy for the individual reaches analyzed was determined based on the results of the soil borings contained in this report.

43. Floodside levee stability analyses require the proposed levee protection be placed at varying distances from the existing canal bank. Since composite sections were used for the stability analyses, the required levee centerline setback from the el -5.0 contour is indicated on each levee stability analysis.

44. Underseepage. Seepage beneath the proposed all-earthen levee sections in Reaches IV (West) and V has been analyzed by a flow net construction. Our flow net construction is based on a levee base width of 80 feet at el 0.0 retaining a floodside water elevation to el 11.6. (Pervious, homogeneous and isotropic foundation conditions were assumed to approximate el -60 or the top of the Pleistocene formation.) Our analyses indicate that the exit gradient at the landside toe of the levee is approximately 0.25. Based upon an escape gradient of approximately 0.25, the factor of safety against piping potential at the landside toe of the levee is approximately 4.0. This is an acceptable factor of safety for the foundation materials at the project site and Eustis Engineering Company recommends that no special measures be undertaken to cut off underseepage beneath the all-earthen levees in Reach IV (West) and Reach V.

45. Eustis Engineering Company has taken borings along the centerline of the canal in an effort to determine the near surface materials on the canal bottom. These borings indicate that, in some areas, pervious sand strata that underlie the project site are directly exposed to canal stages. In most areas, clay cover ranging from 1 to 5 feet exists. Piezometer studies conducted at other hurricane protection project sites along the lakefront indicate that these sand strata even when exposed to the canal stages lag in their reaction to these stages and hydrostatic heads landside of the existing levees do not exceed the existing ground surface. Given the relatively short duration of the design high water and based on data accumulated to date, we have assumed that the hydrostatic uplift forces are at the ground surface on the landside of the proposed flood protection. Our stability analyses are based on this assumption.

46. We recommend that the piezometers installed at the project site be read on a periodic basis, a staff gauge installed at the project site and correlations between the piezometric levels in the foundation sand strata be established with levels in the London Avenue Canal. These are necessary in order to project piezometric levels during stages and to verify the assump-

tions contained in this report. It is pointed out that if this study would indicate the necessity for pressure relief measures the data accumulated would be valuable in minimizing the costs of installation of a system.

47. It is pointed out that our assumptions relative to hydrostatic forces and underseepage assume that a canal bottom will not be dredged. London Avenue Canal is subject to siltation and the permeability of the near surface materials on the canal bottom may be reduced due to the siltation. Removal of these materials may directly expose more pervious materials potentially increasing hydrostatic forces landside of the proposed levee toe. If such dredging occurs or is anticipated, Eustis Engineering Company should be notified in order that our analyses can be refined.

Sheetpile Wall Analyses

48. Cantilever Analyses. Figures 5 and 9 indicate the critical "S" case cantilever analyses for the proposed sheetpile floodwalls along the proposed flood protection alignment. These analyses assume a factor of safety of 1.5 factored into the soil shear strength parameters in order to determine the required penetration of the sheeting. A factor of safety of 1.0 was used to determine the maximum anticipated bending moment.

49. Figure 5 indicates the required penetration and moment diagrams for combination levee fill and sheetpile wall flood protection. An approximate 7-ft height was assumed for the sheetpile in order to minimize the required sheetpile section and penetration. Reach I (Station 0.0 to Station 21+00) right-of-way restrictions preclude the placing of the required fill. For this reason, we have analyzed an alternative sheetpile wall design presented on Figure 8. This alternative assumes the sheetpile wall is driven in areas where the existing grade is at or above el 5.0 and conforms to the minimum section indicated on Figure 9.

50. Settlement. Eustis Engineering Company has computed the settlement of the proposed earth fill and sheetpile

wall flood protection sections. The anticipated settlements are indicated on the table below. Also indicated on this table are the recommended overbuild grades necessary to maintain an approximate 7-ft unsupported height for the sheetpile wall.

Reach	Location	Settlement In Inches
I	Sta. 0+00 to 21+00	8 to 10
I	Sta. 0+00 to 21+00 (Alternate Design)	Negligible
II	Sta. 21+00 to 37+00	Negligible
III	Sta. 37+00 to 90+00	3 to 9
III	Sta. 90+00 to 120+00	6 to 12
IV	Sta. 120+00 to 127+00 (Eastside)	6 to 12
	Sta. 120+00 to 127+00 (Westside)	Negligible
V	Sta. 127+00 to Lake	Negligible

Due to land restrictions in Reach I the recommended grade does not incorporate anticipated settlement. The recommended sheet-pile penetration has been increased accordingly. Also, the alternative design for Reach I assumes that the levee will be constructed with little or no fill in those areas where existing fill can provide the earth section indicated on the analyses. As outlined in Paragraph 40, placement of fill adjacent to existing structures may cause settlement of these structures. Final alignment plans should be furnished to Eustis Engineering Company.

51. Stability Analyses. Stability analyses assuming a failure of the sheetpile wall to the landside and failure of the existing bank to low water are indicated on Figures 6 through 9. Shear strengths and stratigraphy used for these analyses were developed as previously described for levee stability analyses. The landside stability analyses assume the anticipated SWL water elevation to determine the actuating driving forces. The results of these analyses indicate factors of safety at or greater than the minimum 1.3 factor of safety required for design.

52. Underseepage. Underseepage of the recommended sheetpile wall sections was generally evaluated based on Lane's Weighted Creep Ratio Method of Analysis. The results of these computations are tabulated below.

Reach	Weighted Creep Distance In Feet	Net Head In Feet	Lane's Weighted Creep Ratio
I	75	11.9	6.3
I*	46	11.9	3.8
II	56	14.9	3.8
III	57	17.9	3.2
IV (Eastside)	50	16.6	3.4
IV (Westside)	26	11.6	2.3
V**	26	11.5	2.3
V***	57	11.5	4.9

* Alternative Design

** Sta. 127+00 to 144+50 (Eastside);
Sta. 127+00 to 142+50 (Westside)

*** Sta. 147+50 to Lake (Eastside);
Sta. 145+00 to Lake (Westside)

Assuming a safe weighted creep ratio of approximately 2 to 3 for soft to medium stiff clays, penetrations required for cantilever stability need not be increased for Reaches I through IV and for the east side of Reach V. However, borings at the west side of Reach V indicated that the existing levee in some areas may be founded on sand and silty sand materials. The values for Lane's Weighted Creep Ratio in this reach are not acceptable considering these types of materials. Computations indicate that excessive cutoff penetrations are required in order to satisfy Lane's empirical criteria. Therefore, we have analyzed the required cutoff within this reach based on Harr's Method. Harr's Method is a method developed for sheetpile walls incorporating the results of flow net analyses. Based on this method and neglecting the effects of levee fill materials, the required penetration is approximately to el -13.0.

53. Sheetpile Recommendations. Based on the analyses described above, the following table summarizes the recommended sheetpile penetration and bending moments.

Reach	Location	Recommended Tip Elevation In Feet MSL	Recommended Bending Moment In Ft-Lb./Ft. F.S. = 1.0
I	Sta. 0+00 to 21+00	-34	23192
I	Sta. 0+00 to 21+00	-42	35970
II	Sta. 21+00 to 37+00	-20.5	14964
III	Sta. 37+00 to 120+00	-20	15506
IV (East)	Sta. 120+00 to 127+00	-21	15667
IV (West)	Sta. 120+00 to 127+00	-1	1945
V (East)	Sta. 127+00 to 147+50	-1	1945
V (East)	Sta. 147+50 to Lake	-16	18205
V (West)	Sta. 127+00 to 142+50	-13	1945
V (West)	Sta. 142+50 to 145+00	-1 to -16*	1945 to 18205*
V (West)	Sta. 147+50 to Lake	-16	18205

*Transition

54. Sheetpiles should be driven with a single acting air or steam hammer delivering between 8000 and 16,000 ft-lb of energy per blow. Consideration should be given to the use of a vibratory hammer. However, buried wood encountered at many boring locations may preclude the efficient use of a vibratory hammer in Reach III.

Bridge Modifications

55. Allowable Pile Load Capacities. The recommended allowable pile load capacities for various lengths and sizes of square precast prestressed concrete piles and 14-in. diameter pipe piles are tabulated on Figures 10 through 37. These

allowable pile load capacities contain a factor of safety of 2 against actual failure of the pile through the soil. Both tension and compression load capacities are provided for piles with butts at the existing grade crossing and at the elevation of the canal bottom. Pipe pile capacities are provided to evaluate the existing bridge foundations.

56. Pile Driving. It is recommended that the piles be driven with a steam or air hammer delivering approximately 19,500 ft-lb of energy per blow. In order for these piles to penetrate the sand strata encountered between approximate el -10 and -45 at Boring 15 and Boring 50, -12 and -45 at Borings 19 and 53, and -16 and -15 at Borings 27, 29, 61 and 63, predrilling may be required. Also, if piles are driven in the levee cross-section, it may be desirable to predrill in order to minimize the lateral dispacement of soils as well as to minimize the build up of excess pore pressures due to pile driving. If predrilling is required, it should be accomplished by a wet rotary method utilizing a fishtail bit. The diameter of the predrilled hole should not exceed 75 percent of the side dimension of the square pile. The depth of the predrilling operations should extend to no more than 5 feet below the bottom of the sand strata. Close field supervision must be maintained by experienced personnel to insure that proper procedures are followed and accurate records are kept on all piles.

57. Past experience indicates that pile driving operations may transmit vibrations to adjacent structures, particularly when piles are to be firmly seated or driven through a sand stratum with a high driving resistance. In addition, vibrations generated by pile driving operations may densify loose sand stratum resulting in settlement of existing structures. Also, surface waves propagating through soft organic soils may also cause damage to existing structures. A study should be made to determine the tolerance of existing structures to vibratory loads and settlements. Eustis Engineering Company is available to monitor vibrations during all pile driving operations and can provide consultation concerning the effect of vibrations on existing structures.

58. <u>Test Piles and Pile Load Tests.</u> It is recommended that at least one test pile of the type anticipated for final design be driven at each bridge site location to give a general indication of the expected driving resistances throughout the project site. These test piles should be driven with the same type of equipment and techniques that will be used to drive the job piles. The test piles will provide valuable information regarding the expected driving resistances and vibrations that may be anticipated during the driving of the job piles. At least one pile should be load tested to verify the estimated design load capacities contained in this report. The pile showing the least resistance to driving should be the one selected for the pile load test. The pile should be load tested to failure in accordance with ASTM D 1143. The U.S. Army Corps of Engineers has standardized specifications outlining load increments and load cycling. Eustis Engineering Company recommends that the load increments past the design load be one-half the increments recommended by the ASTM specification.

59. Eustis Engineering Company will be available for discussions regarding the formulation of a test pile program, and can provide personnel for the logging of the test piles, application of the loads and evaluation of the results of the load tests. We can also log the driving of the job piles as well as evaluate the integrity of the job piles based on the driving logs.

60. <u>Estimated Settlement.</u> It is estimated that the settlement due to imposed structural loads on the pile lengths recommended in this report for use at the bridge modifications will be small and on the order of 0.25 to 0.5 of an inch. Our settlement analyses assume that the bridge modifications are supported by widely spaced single rows of piles or by isolated groups of piles not exceeding four piles per group. Analyses assume that little or no fill is needed. If fill in excess of 2 feet is required at the bridges or pile group dimensions other than cited above are proposed, Eustis Engineering Company should be notified in order that our settlement analyses can be refined.

Gated Structures

61. Deep Seated Stability Analyses. Based on criteria supplied to Eustis Engineering Company, we have evaluated the potential for a deep seated stability failure of pile supported floodwall and gated structures. Results of these analyses are indicated on Figure 38. These analyses indicate that the active driving forces for all failure surfaces analyzed do not exceed the summation of the resisting forces and the passive driving forces. Therefore, there is no potential for deep seated stability failure beneath the floodwalls or gated structures.

62. Based on Lane's Weighted Creep Ratio of 3.0, it is recommended that the sheetpile cutoff beneath the gated structures be extended to 15 feet below the base of the structure. As these sections are not required to carry any bending moments, these may be straight web or shallow arched sections.

63. Allowable Pile Load Capacities. Recommended allowable pile load capacities for various lengths and sizes of square precast concrete piles or pipe piles indicated on Figures 10 through 19 are applicable to the floodwall and gated structures in Reach I. Interpolation of allowable pile load capacities between bridge sites is recommended for the floodwall structures. Please refer to Paragraphs 55 through 58 for recommendations regarding pile driving, predrilling, vibratory loads, test piles and pile load tests.

64. Lateral Loads. Eustis Engineering Company understands that batter piles will be used to resist lateral loads. Distribution of loads to the piles must be analyzed by the Hrenicoff Method of Analysis which requires a coefficient of horizontal subgrade reaction (k_h) as a design parameter. Based on the results of in situ field tests and laboratory test data, we have computed k_h and have plotted its general variation with depth in Reach I on Figure 39. Horizontal reactions to batter piles should be determined by resolving the horizontal component of the force polygon as determined by the pile batter.

65. The concrete piles used to support gated structures and floodwalls in Reach I derive the majority of their supporting capacity primarily through skin friction and should be investigated to determine the effects of group action if piles are driven in groups. The supporting value of these piles should be investigated on the basis of group perimeter shear by the expression shown on Figure 40. The minimum center to center spacing of the piles should be as indicated on Figure 40.

66. Estimated Settlement. It is estimated that settlement due to imposed structure loads on the pile lengths greater than 45 feet will be small and on the order of 0.5 to 0.75 of an inch. Our settlement analyses assume little or no fill is placed at the gate. If fill in excess of 2 feet is proposed for use at the gated structures or floodwalls, Eustis Engineering Company should be notified in order that our settlement analyses can be refined.

Construction Recommendations

67. Earth Work. Eustis Engineering Company recommends that site preparation, levee fill and compaction be accomplished in accordance with the Department of the Army, Mississippi River Commission, Lower Mississippi Valley Division, Corps of Engineers Standard Specifications for Levee Construction. Levee fill should be either a CH or CL material as classified by the Unified Soil Classification System and compacted by semi-compaction methods. Material for levee fill should be compacted within the following moisture content ranges.

Material	Moisture Content Minimum	Moisture Content Maximum
CL	18	32
CH	20	50

It is the intent of these specifications to construct a relatively uniform embankment free of large gaps, voids and loose materials. To accomplish this, backfill should be spread in 8 to 10-in. lifts and each lift compacted with a minimum of three passes of a D5 dozer or equivalent. When proper compaction has been achieved, a D5 dozer should be able to "walk out" without fill material sticking to the treads or otherwise disturbing the lift. If this cannot be achieved, moisture control, such as disking to dry back the material or spraying to wet the material, may be required.

68. Existing Utility Lines. Eustis Engineering Company understands that during recent high water stages in Lake Pontchartrain flow through an abandoned utility pipeline was exiting landside of the flood protection. Eustis Engineering Company recommends that this pipeline be at least plugged with a bentonite-cement slurry and should be observed after being plugged to determine if seepage is still occurring along the pipe. An effort should be made to determine if any other abandoned lines are in the area and these should be similarly plugged and observed.

EUSTIS ENGINEERING COMPANY

By____________________________
Lloyd A. Held, Jr.

W. W. Gwyn:bh

EEC No. 9223

LOCATION OF BORINGS

SCALE: 1"=40'

- LEVEE BORINGS DRILLED 3 OCTOBER - 10 DECEMBER 1985
- CANAL BORINGS DRILLED 13 NOVEMBER - 17 DECEMBER 1985
- BORINGS NOT TAKEN DUE TO CONCRETE CANAL LINER.

BORINGS B-45, B-56 & B-65 ARE 5" DIA. UNDISTURBED BORINGS. ALL REMAINING UNDISTURBED BORINGS ARE 3" DIA.

- USACE BORINGS TAKEN 23 OCTOBER THROUGH 25 OCTOBER 1985

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

LOCATION OF BORINGS

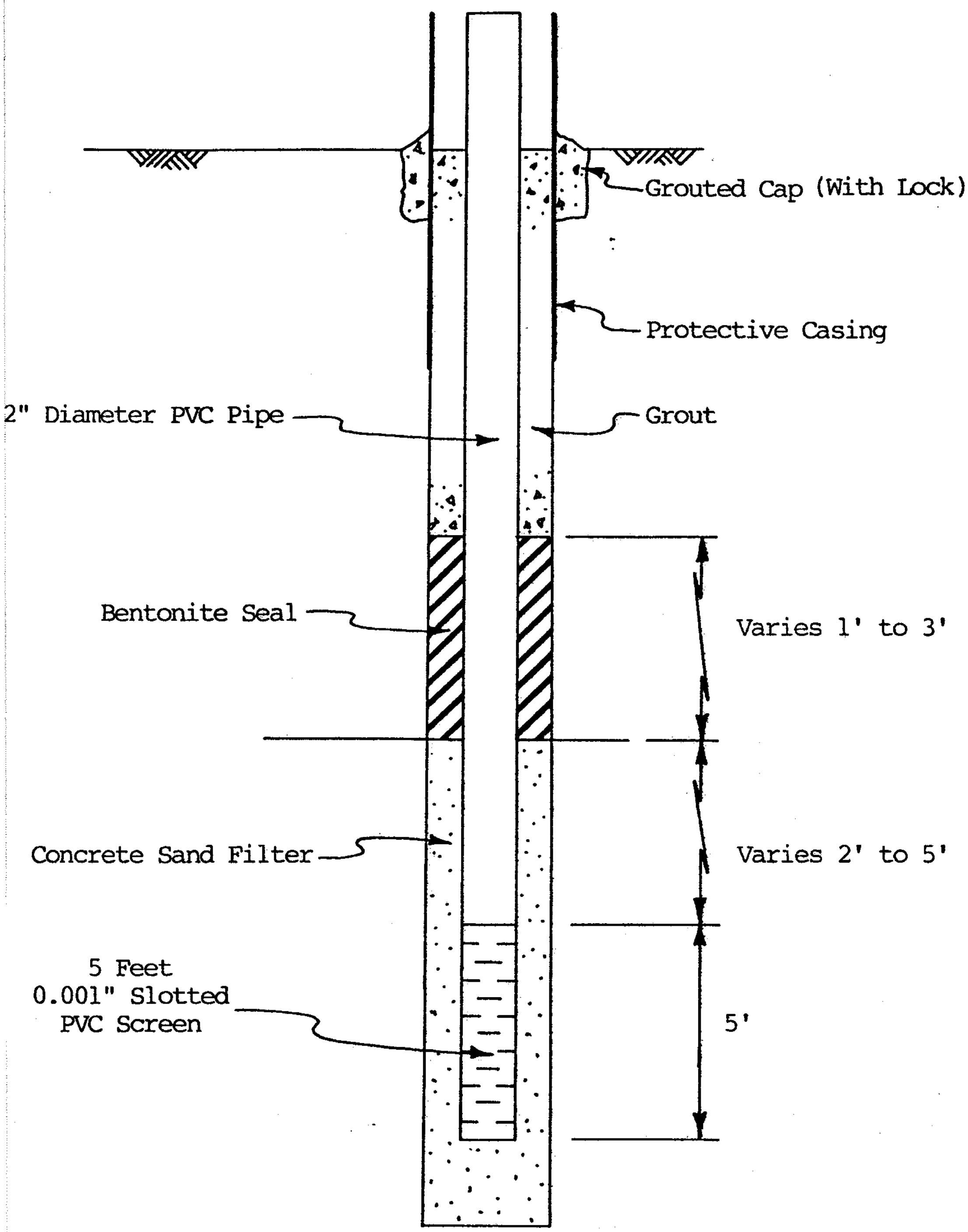
FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

[redacted]

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 1



NOT TO SCALE

TYPICAL PIEZOMETER DETAIL

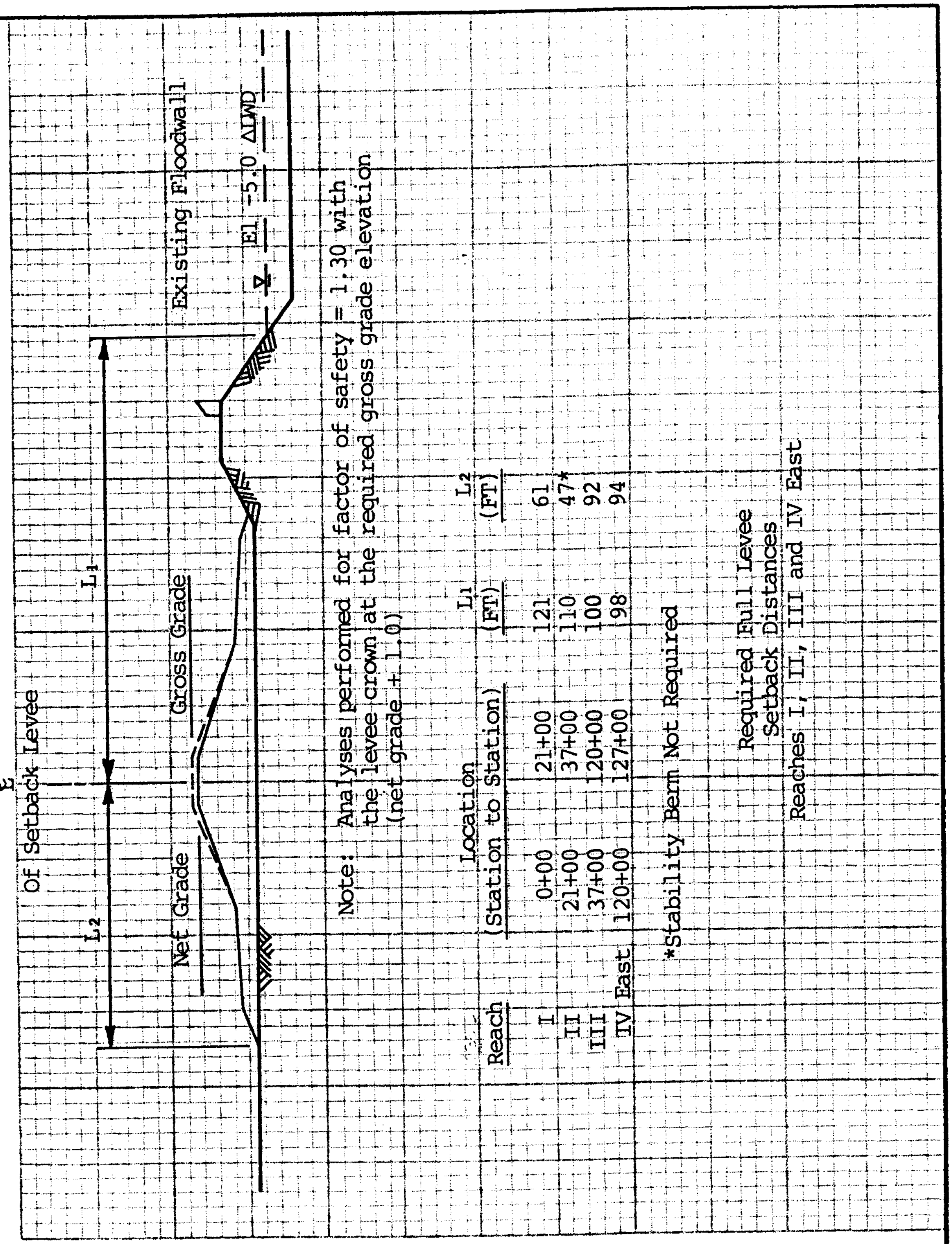
Geotechnical Investigation
Orleans Levee District
London Avenue Outfall Canal
OLB Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

Fig. 2

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LOUISIANA



Note: Analyses performed for factor of safety = 1.30 with the levee crown at the required gross grade elevation (net grade + 1.0)

Reach	Location (Station to Station)		L_1 (FT)	L_2 (FT)
I	0+00	21+00	121	61
II	21+00	37+00	110	47*
III	37+00	120+00	100	92
IV East	120+00	127+00	98	94

*Stability Berm Not Required

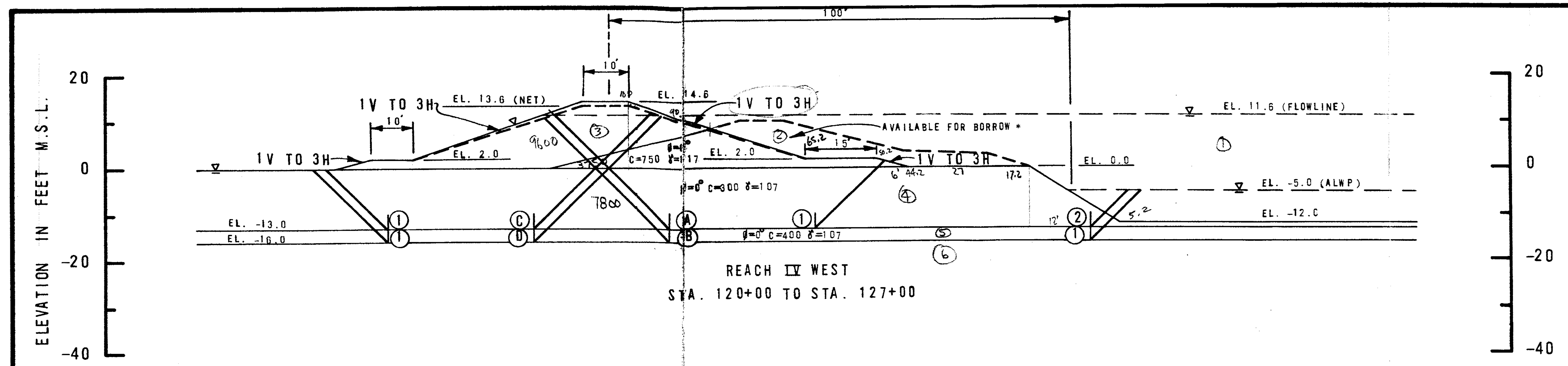
Required Full Levee Setback Distances
Reaches I, II, III and IV East

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

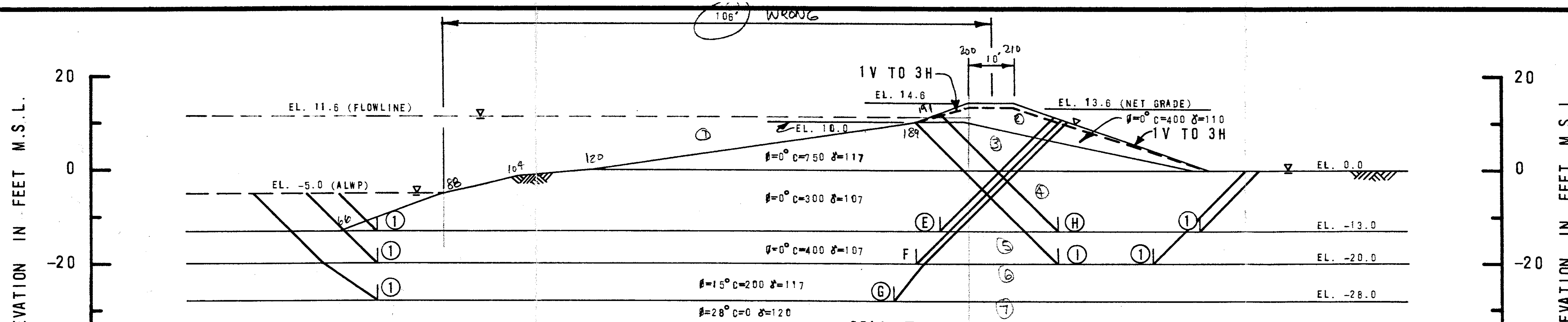
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 3



REACH IV WEST

STA. 120+00 TO STA. 127+00

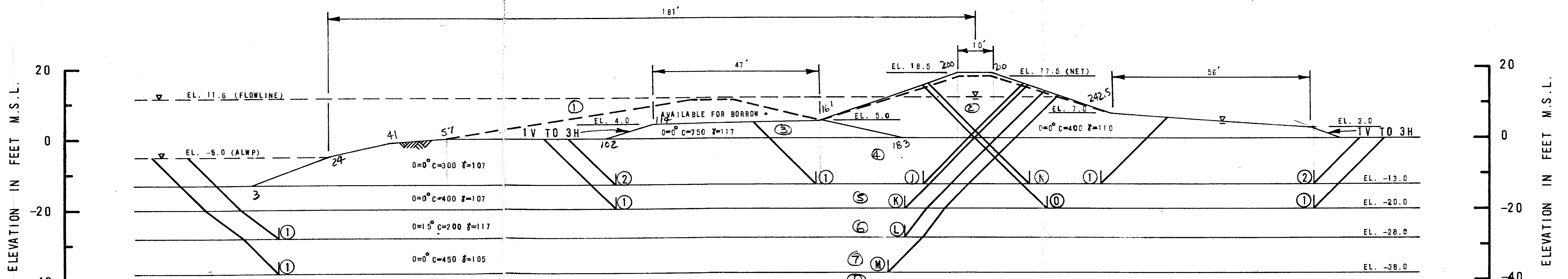


REACH V

STA. 127+00 TO STA. 144+50 EAST SIDE

STA. 127+00 TO STA. 142+00 WEST SIDE

SLIP SURFACE			DRIVING FORCE			RESISTING FORCE				FACTOR OF SAFETY ΣR/ΣD
NUMBER		ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	
A	1	-12 -13	39566	11778	27788	19391	9300	7800	36491	1.31
A	2	-12 -13	39566	2354	37212	19391	27000	1835	48226	1.30
B	1	-16	48210	4662	43548	20732	34964	3485	59181	1.36
C	1	-12 -13	38969	10580	28389	20145	9300	7800	37245	1.31
D	1	-16	47775	15234	32541	22679	12400	10200	45279	1.39
E	1	-13	39561	2114	37447	21354	36000	1159	58513	1.56
F	1	-20	60779	8426	52353	26218	46000	5600	77818	1.49
G	1	-28	90726	22175	68551	36618	75434	12951	125003	1.82
H	1	-13	39773	9120	30653	24422	9000	7800	41222	1.34
I	1	-20	61373	24230	37143	28426	7960	13400	49786	1.34
J	1	-13	50526	9012	41514	20220	25800	7693	53713	1.29
J	2	-13	50526	17571	32955	20220	9000	14282	43502	1.32
K	1	-20	74014	21195	52819	25427	32360	13226	71013	1.34
L	1	-28	107433	22175	85258	36461	78715	12951	128127	1.50
M	1	-38	157672	49371	108301	45052	76500	21729	143281	1.32
N	1	-13	50297	20857	29440	19946	6000	12492	38438	1.31
N	2	-13	50297	10215	40082	19946	24000	7800	51746	1.29
O	1	-20	73693	22575	51118	25107	29960	13400	68467	1.34



REACH V

STA. 147+50 TO LAKE PONTCHARTRAIN EAST SIDE

STA. 145+00 TO LAKE PONTCHARTRAIN WEST SIDE

* DEGRADING OF EXISTING SECTIONS IN NOT REQUIRED. FLOODSIDE ANALYSES ON FIGURES 6 & 7 INDICATE AN ADEQUATE FACTOR OF SAFETY FOR EXISTING SECTIONS.

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER CU. FT.
C=COHESION IN POUNDS PER SQ. FT.
Φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZOMETRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

EARTH LEVEE STABILITY ANALYSES

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 4

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

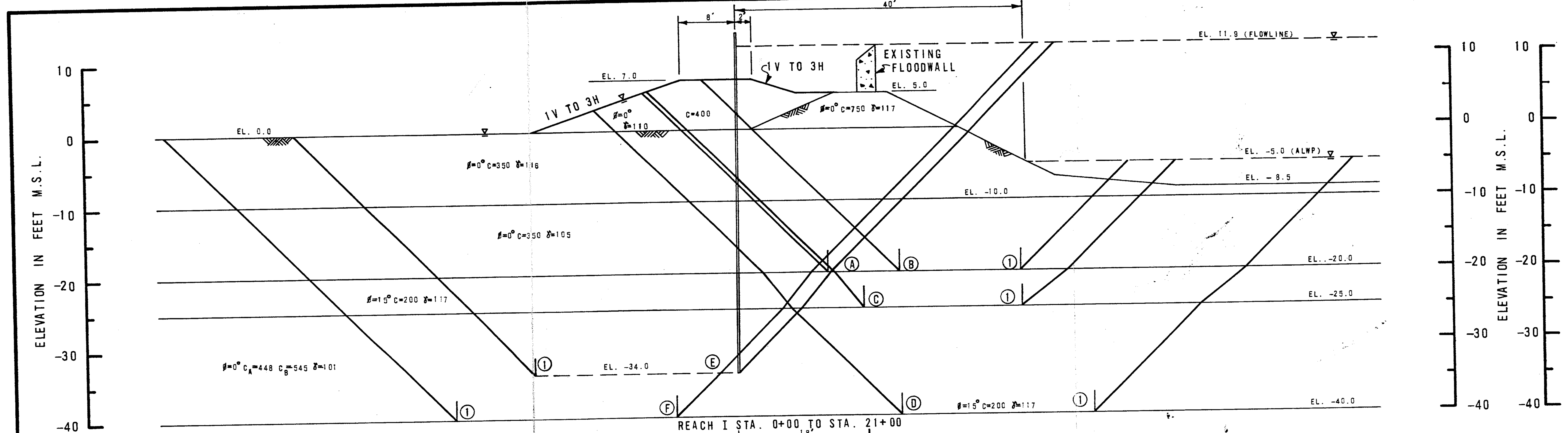
S CASE
CANTILEVER SHEET PILE ANALYSES

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

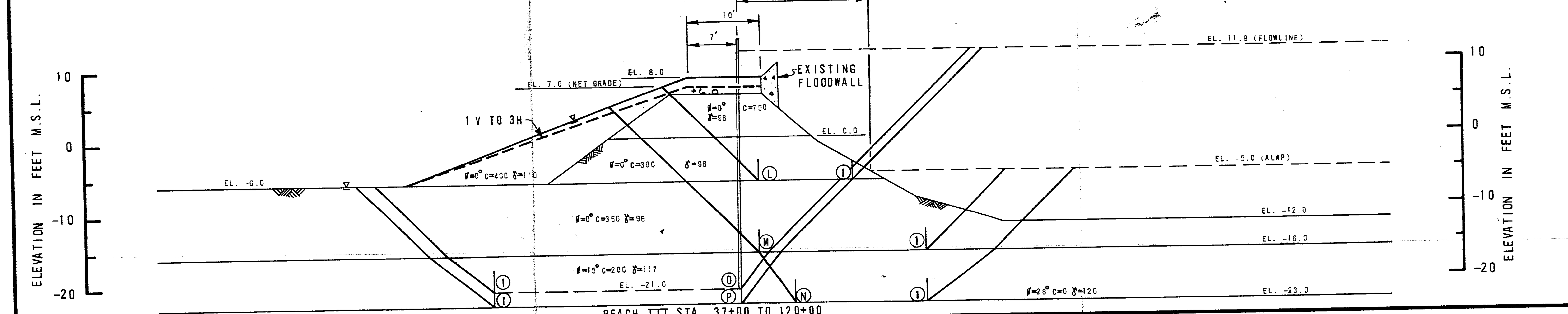
BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

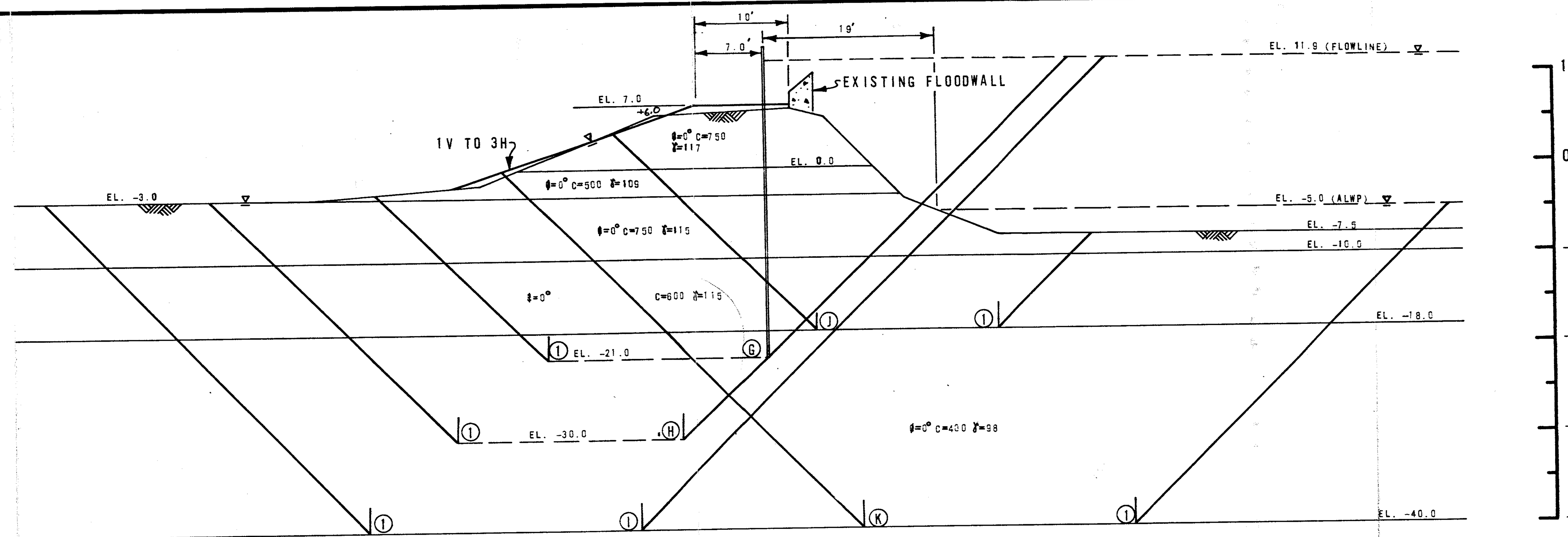
FIGURE 5



REACH I STA. 0+00 TO STA. 21+00



REACH III STA. 37+00 TO 120+00



REACH II STA. 21+00 TO STA. 37+00

SLIP SURFACE			DRIVING FORCE			RESISTING FORCE				FACTOR OF SAFETY
NUMBER		ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	ΣR / ΣD
(A)	(1)	-20	39246	11342	27904	18217	9415	8647	36279	1.30
(B)	(1)	-20	38013	11342	26671	19990	5950	8647	34587	1.30
(C)	(1)	-25	54677	19991	34686	24084	7665	13657	45406	1.31
(D)	(1)	-40	115749	60813	54936	35782	14661	26317	76760	1.40
(E)	(1)	-34	105883	68179	37704	26220	13076	29375	68671	1.82
(F)	(1)	-40	130263	89447	41816	32461	16459	33855	82775	1.38
(G)	(1)	-21	47190	22242	24948	21295	10000	23167	54462	2.18
(H)	(1)	-30	79818	42112	37706	28961	10000	29700	68661	1.82
(I)	(1)	-40	122826	75138	47688	34634	12000	37700	84334	1.77
(J)	(1)	-18	34328	176	26152	29561	8000	13350	50911	1.94
(K)	(1)	-40	109459	61891	47568	40328	12000	30950	83278	1.75
(L)	(1)	-6	9966	171	9795	16512	3900	1136	21548	2.20
(M)	(1)	-16	26858	4376	21582	18095	7396	3698	29189	1.35
(N)	(1)	-23	44194	13016	31178	24619	7834	8279	40732	1.30
(O)	(1)	-21	440004	13673	30331	11643	17260	14133	45146	1.38
(P)	(1)	-23	50001	17230	32771	12743	18270	14133	45146	1.38

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER SU. FT.
C=COHESION IN POUNDS PER SQ. FT.
C_A=AVERAGE COHESION IN POUNDS PER SQ. FT. IN VERTICAL REACH OF STRATUM.
C_B=COHESION IN POUNDS PER SQ. FT. ALONG HORIZONTAL PLANE AT BOTTOM OF STRATUM.
Φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZOMETRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

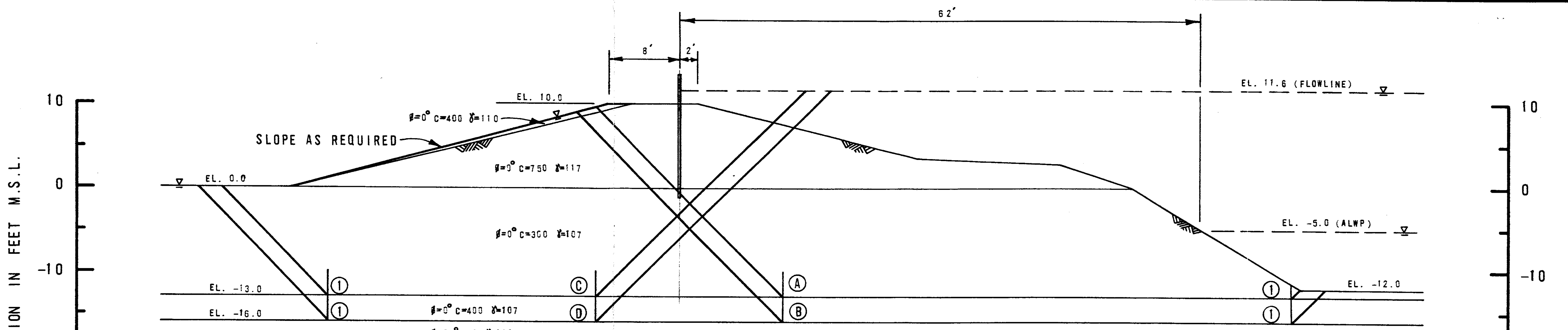
SHEET PILE WALL STABILITY ANALYSES

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

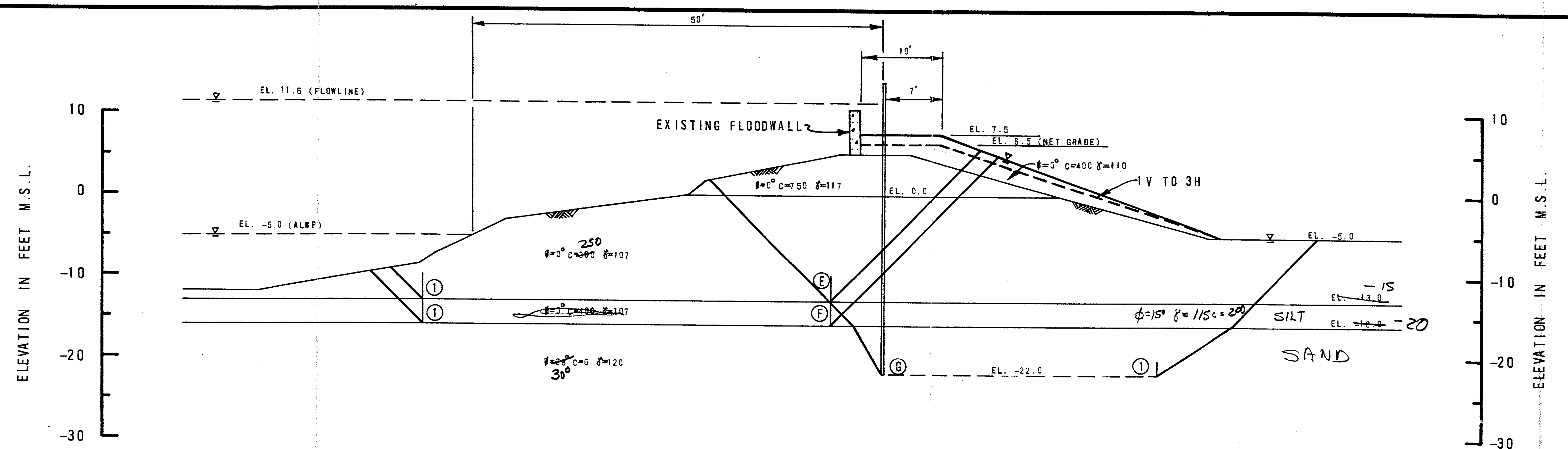
EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 6



REACH IV WEST

STA. 120+00 TO STA. 127+00



REACH IV EAST

STA. 120+00 TO STA. 127+00

SLIP SURFACE			DRIVING FORCE			RESISTING FORCE				FACTOR OF SAFETY
NUMBER		ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	$\Sigma R/\Sigma D$
Ⓐ	①	-13	26933	2035	26898	20014	19800	600	40414	1.50
Ⓑ	①	-16	36339	4150	32189	23010	21718	3000	47728	1.48
Ⓒ	①	-13	30742	9415	21237	19500	9600	7800	36900	1.73
Ⓓ	①	-16	38798	14070	24728	21001	12800	10200	44001	1.78
Ⓔ	①	-13	20836	2310	18526	14315	14700	2087	31102	1.67
Ⓕ	①	-16	27353	4729	22624	15665	18124	4252	38041	1.68
Ⓖ	①	-22	51187	16240	34947	16351	21545	14398	52294	1.50

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER CU. FT.
C=COHESION IN POUNDS PER SQ. FT.
Φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZOMETRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

SHEET PILE WALL STABILITY ANALYSES

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

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SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 7

REACH V

STA. 127+00 TO LAKE PONTCHARTRAIN

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER CU. FT.
C=COHESION IN POUNDS PER SQ. FT.
Φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZOMETRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.

SLIP SURFACE			DRIVING FORCE			RESISITNG FORCE				FACTOR OF SAFETY
NUMBER		ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	$\Sigma R/\Sigma D$
(A)	(1)	-13	32099	2114	29955	23566	31500	1159	56225	1.88
(B)	(1)	-20	52714	8155	44559	27881	42745	5600	76226	1.71
(C)	(1)	-38	128748	54796	73952	46793	33750	22845	103388	1.40
(D)	(1)	-13	32531	9276	23255	21900	10500	7800	40200	1.73
(E)	(1)	-20	53312	21634	31678	25633	14000	13401	53034	1.67
(F)	(1)	-38	130343	78506	51837	39735	13500	32760	85995	1.66

* 1V TO 4H LANDSIDE SLOPE AND LANDSIDE GROUND SURFACE AT EL. 0.0 ASSUMED FOR LANDSIDE ANALYSIS.

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

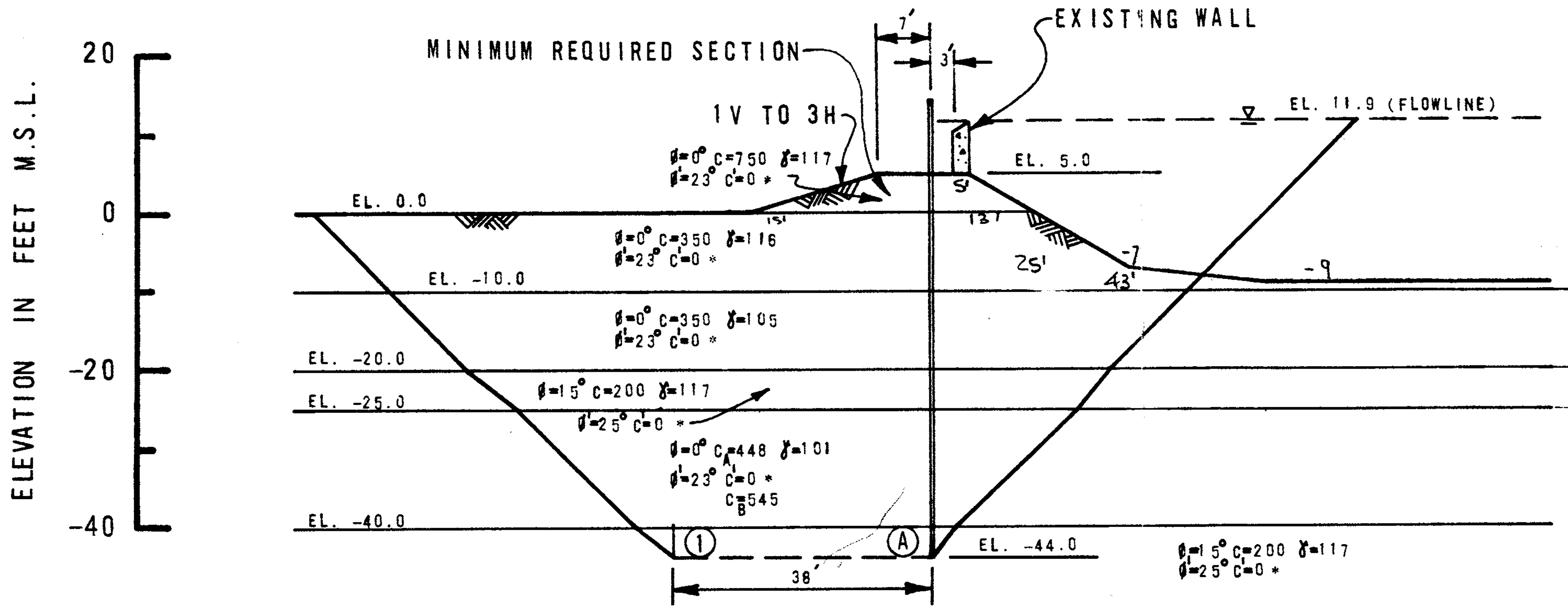
SHEET PILE WALL STABILITY ANALYSIS

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

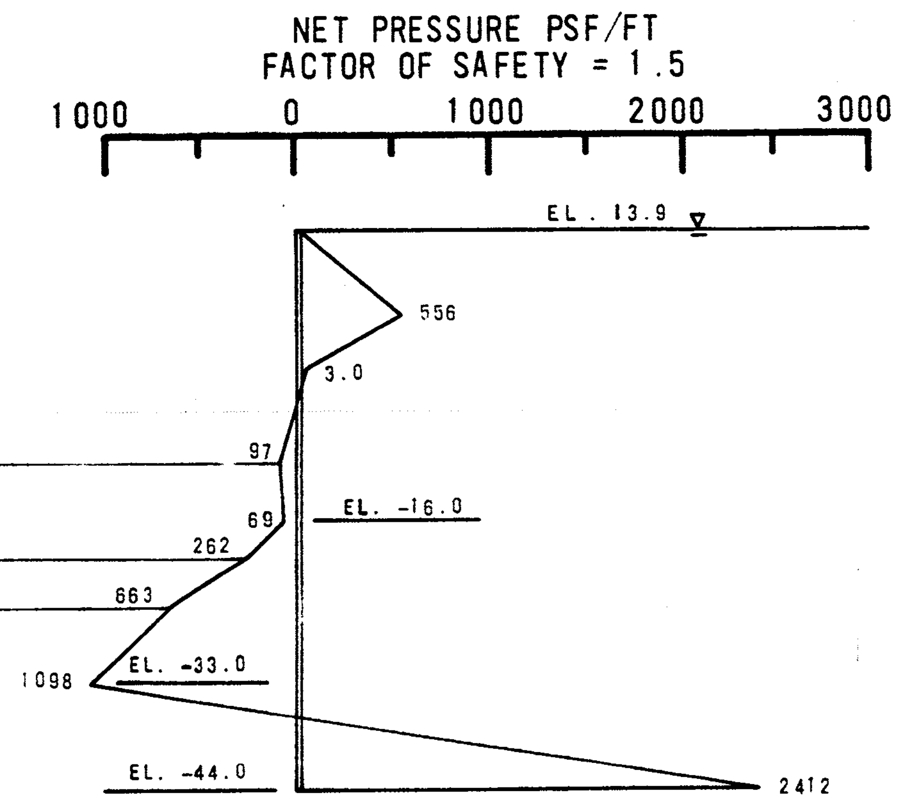
EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

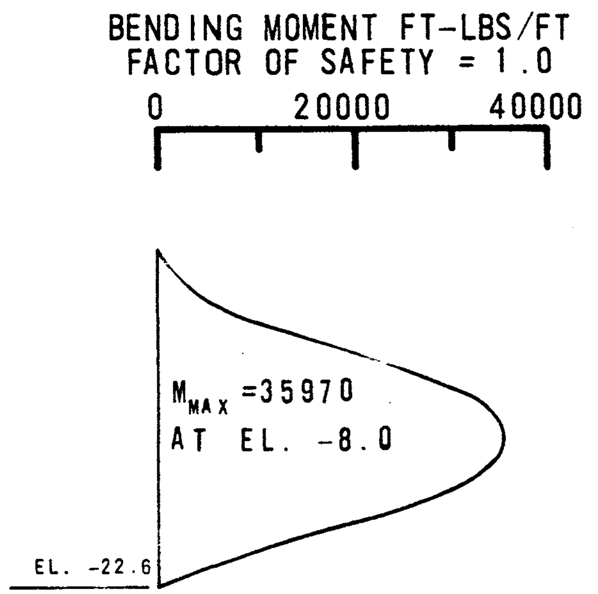
FIGURE 8



STABILITY ANALYSIS

* "S" CASE SHEAR STRENGTH PARAMETERS USED FOR CANTILEVER ANALYSIS.





"S" CASE CANTILEVER

SHEET PILE ANALYSIS

SLIP SURFACE		DRIVING FORCE			RESISTING FORCE				FACTOR OF SAFETY ΣR/ΣD
NUMBER	ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	
(A) (1)	-44	141010	106622	34388	31612	26879	41320	99831	2.90

NOTE: SEE FIGURE 6 FOR FLOODSIDE STABILITY ANALYSES.

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER CU. FT.
C=COHESION IN POUNDS PER SQ. FT.
C_A=AVERAGE COHESION IN POUNDS PER SQ. FT. IN VERTICAL REACH OF STRATUM.
C_B=COHESION IN POUNDS PER SQ. FT. ALONG HORIZONTAL PLANE AT BOTTOM OF STRATUM.
Φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZOMATRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.

GEOTECHNICAL INVESTIGATION
LONDON AVENUE CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

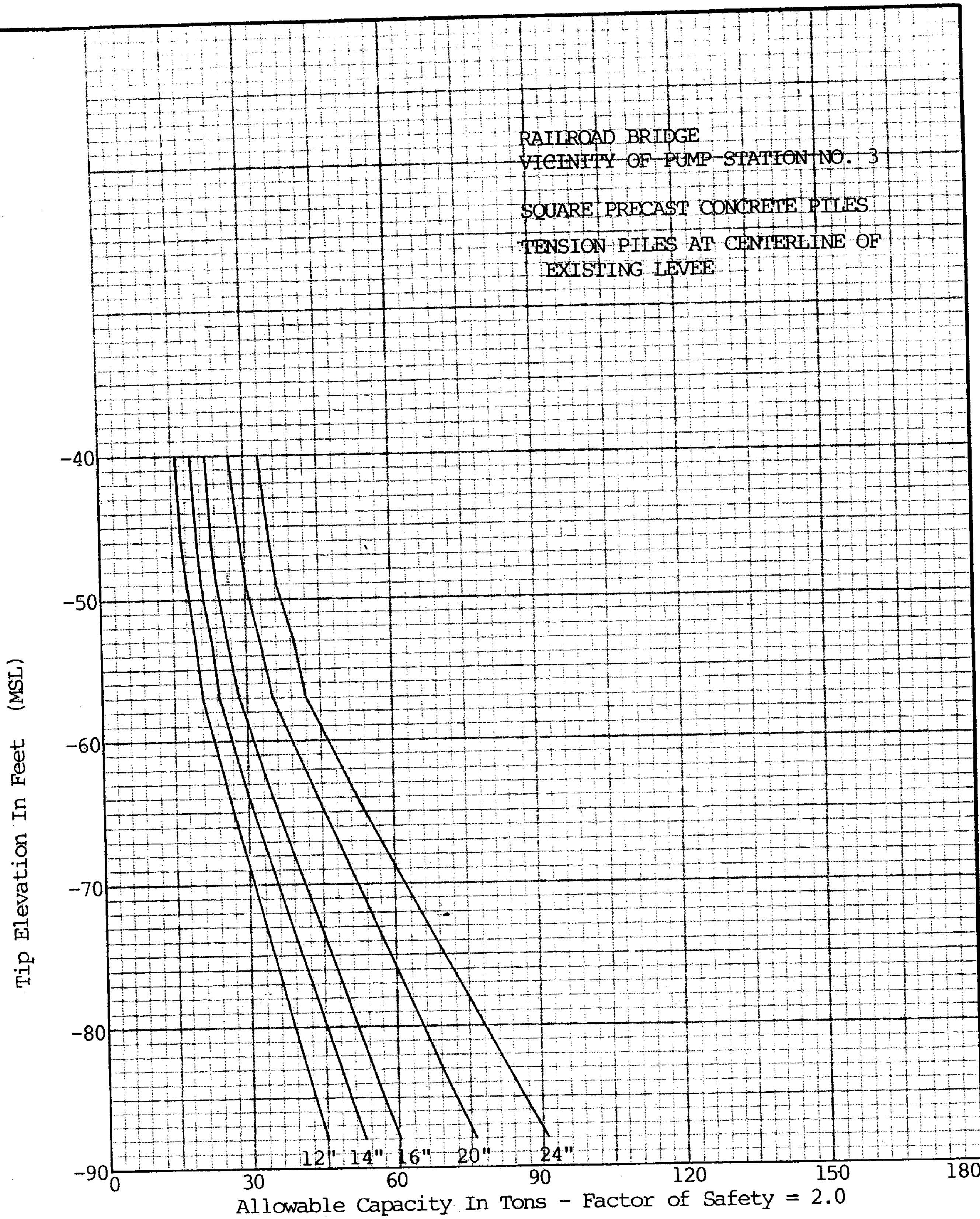
REACH I
STA. 0+00 TO STA. 21+00
ALTERNATE SHEET PILE DESIGN
CANTILEVER SHEET PILE
AND
STABILITY ANALYSIS

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 9



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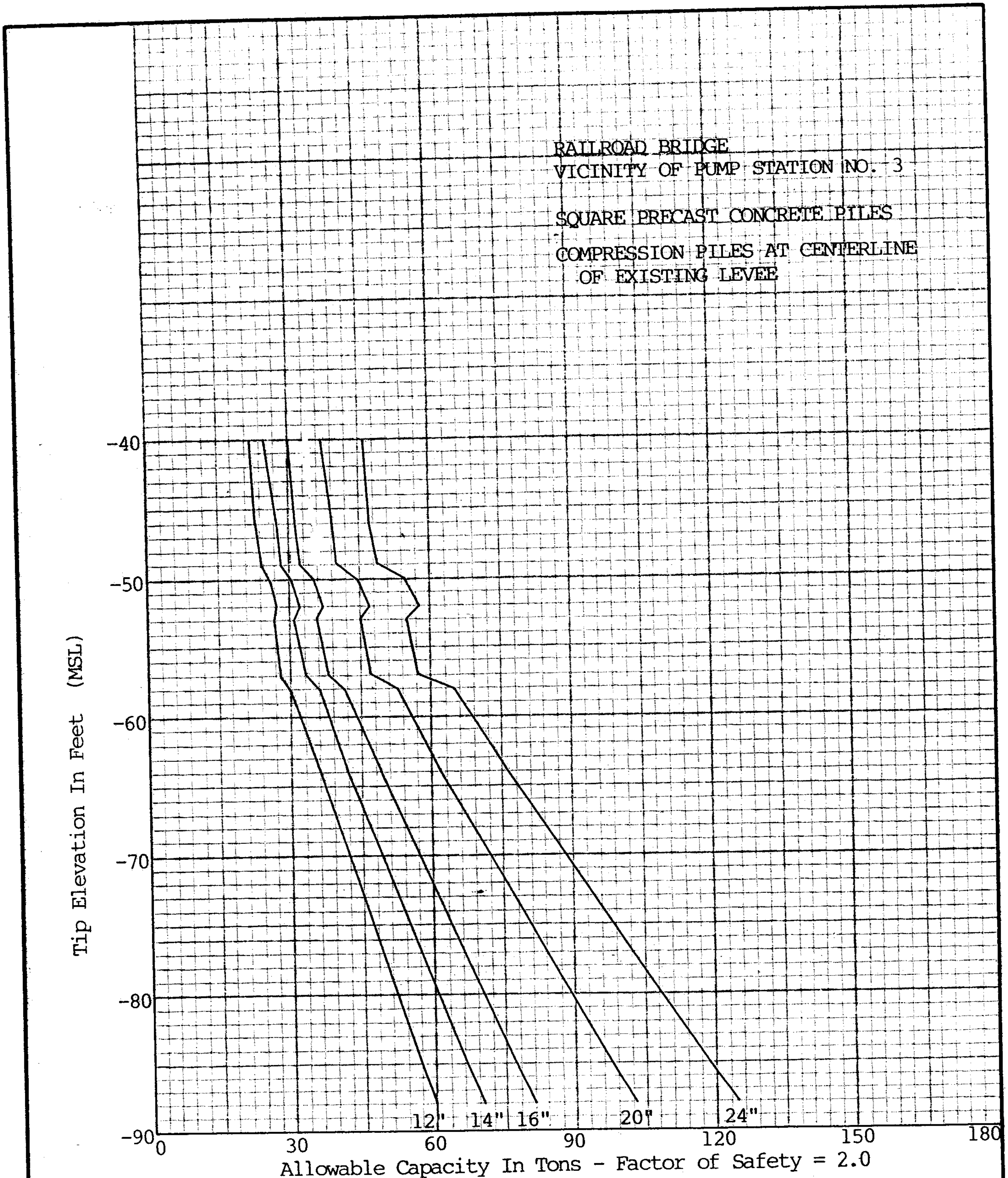
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 10

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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New Orleans, Louisiana
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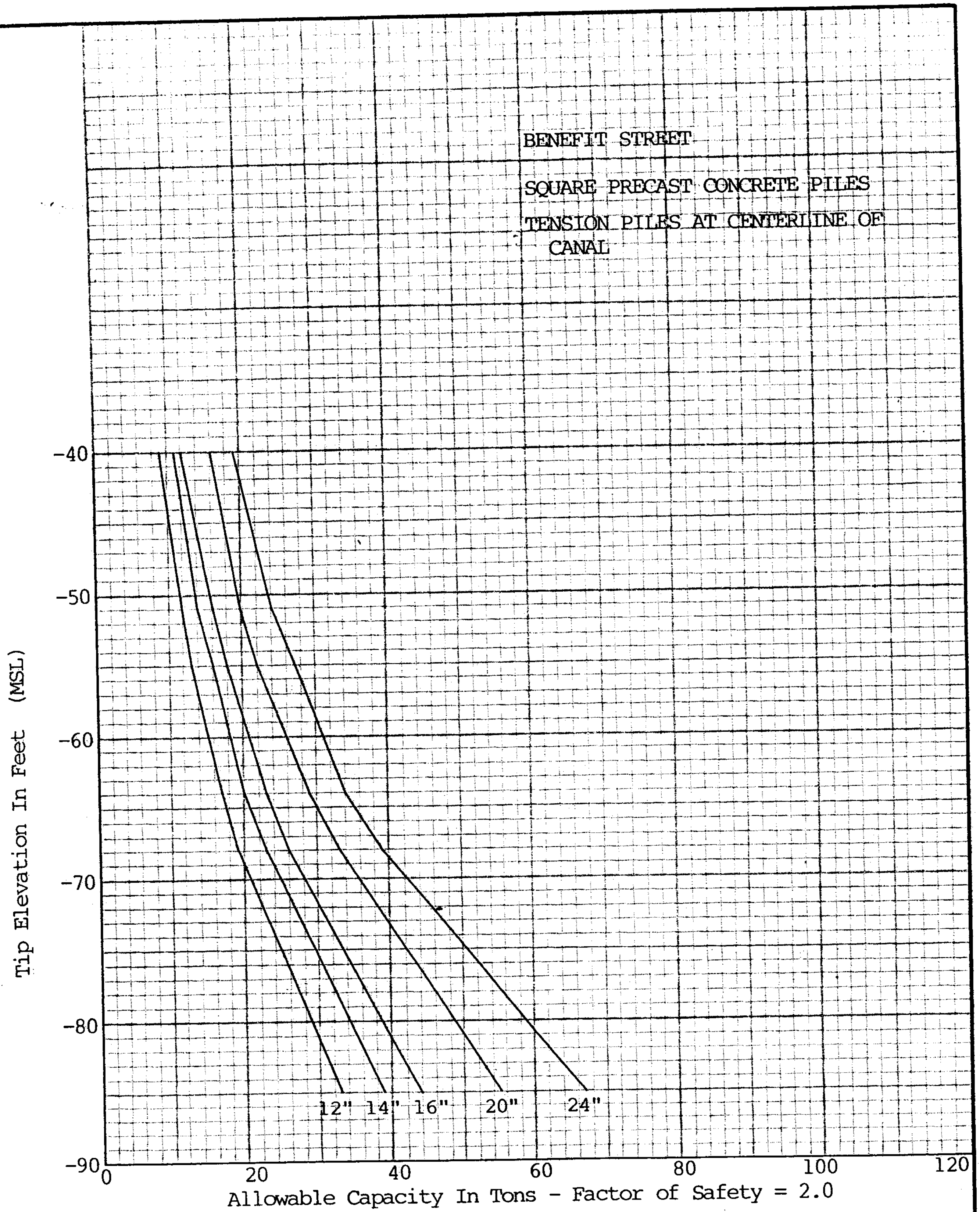
Fig. 11



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New Orleans, Louisiana
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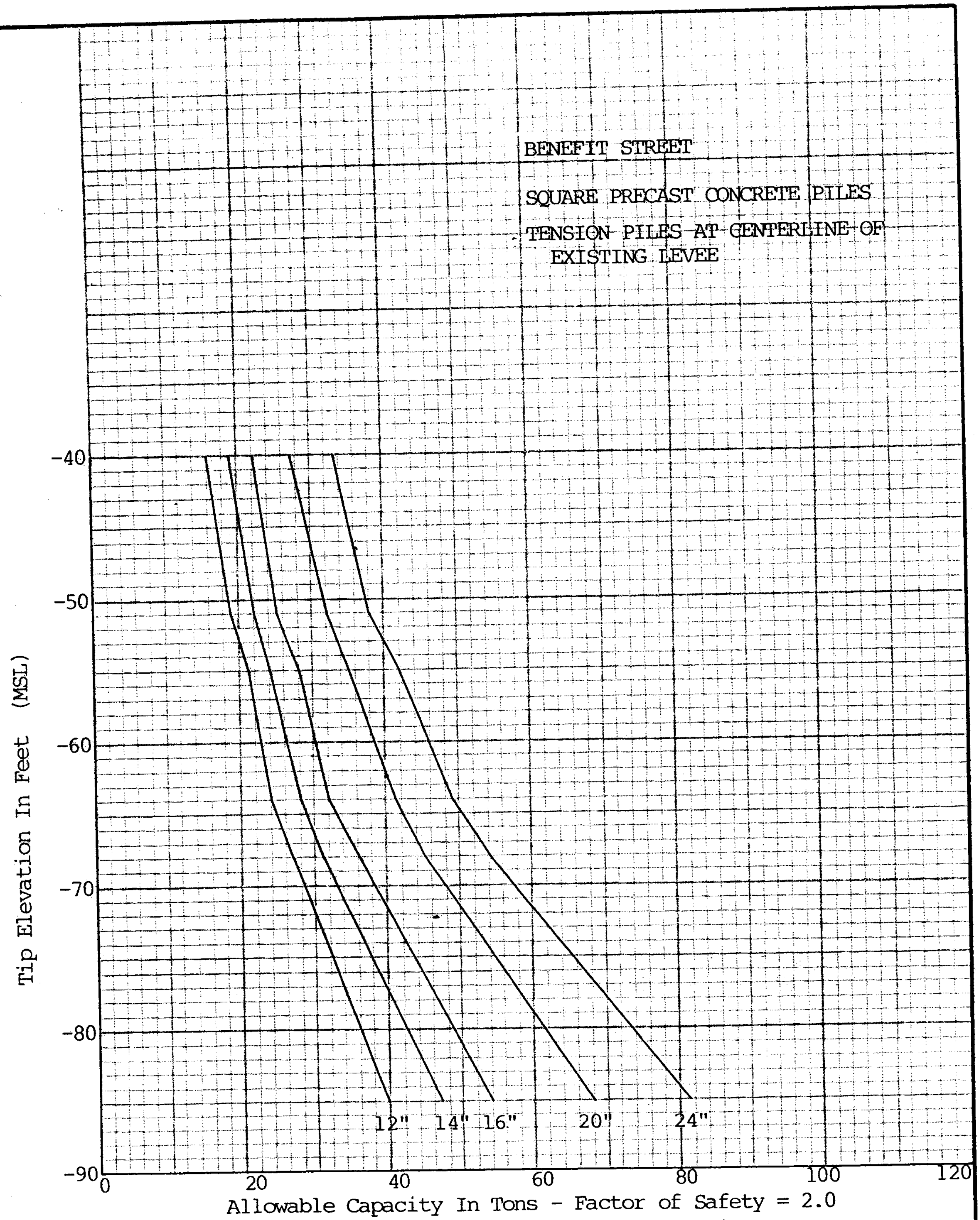
Fig. 12

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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New Orleans, Louisiana
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Fig. 13

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

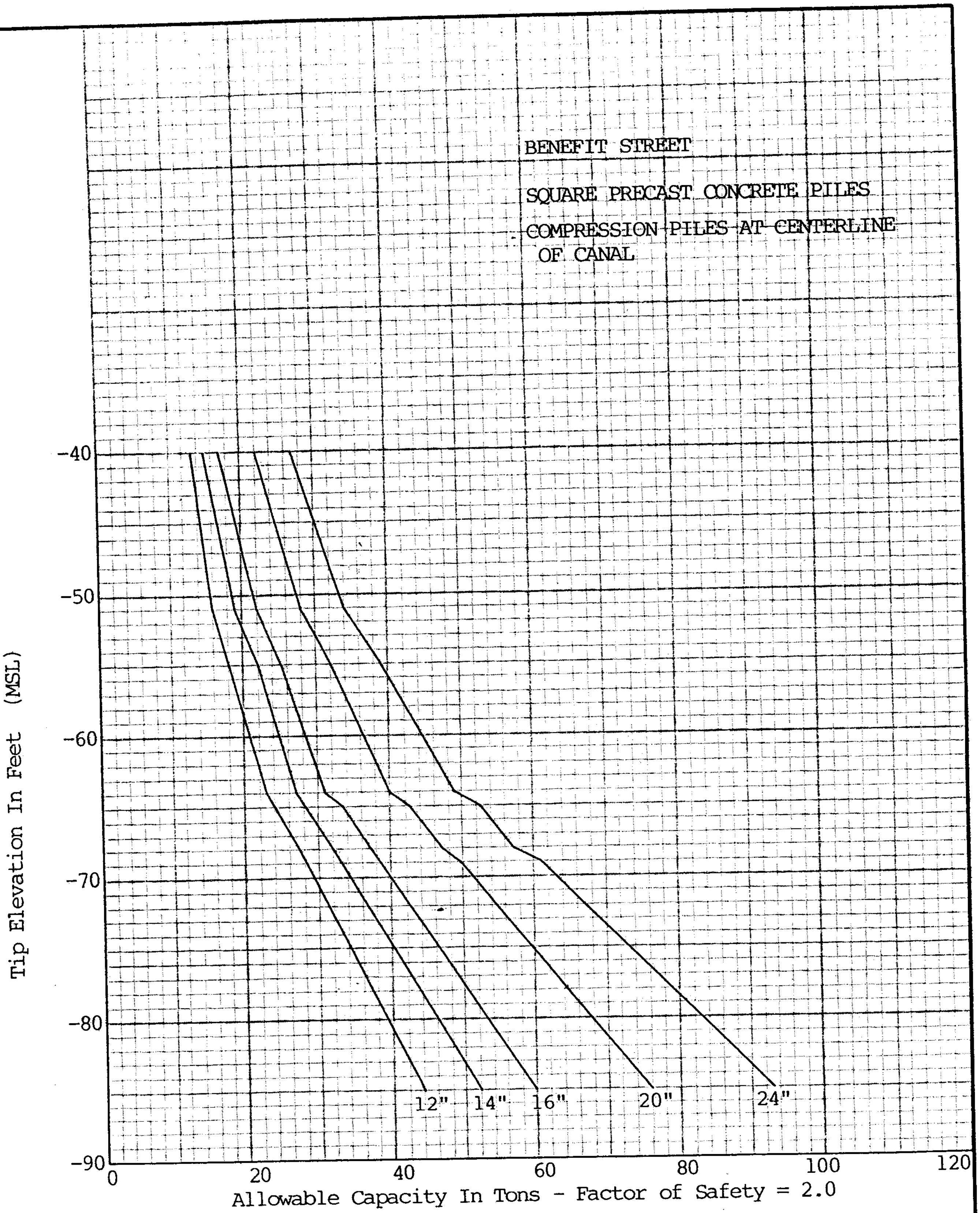

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Fig. 14

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

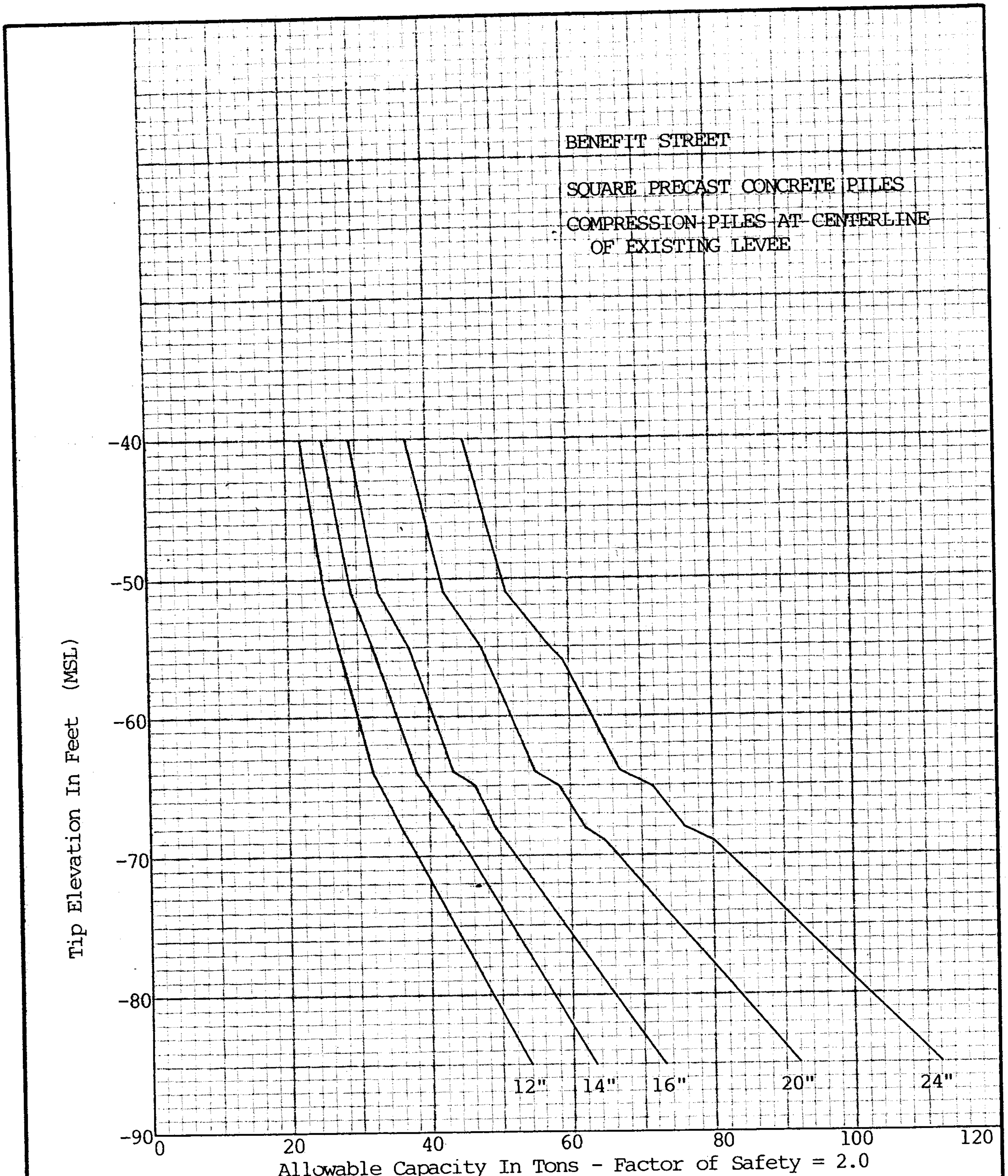

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Fig. 15

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

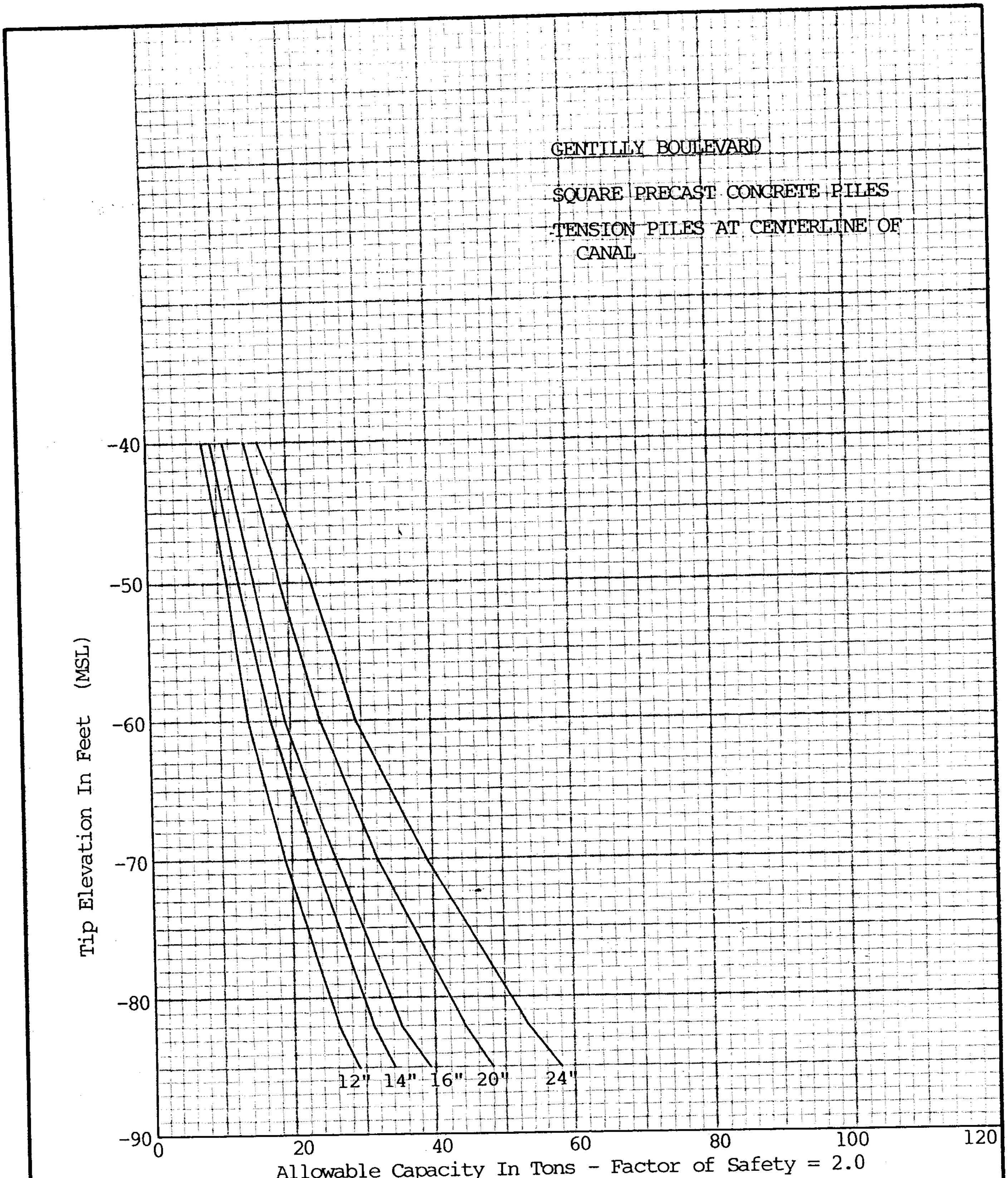

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Fig. 16

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

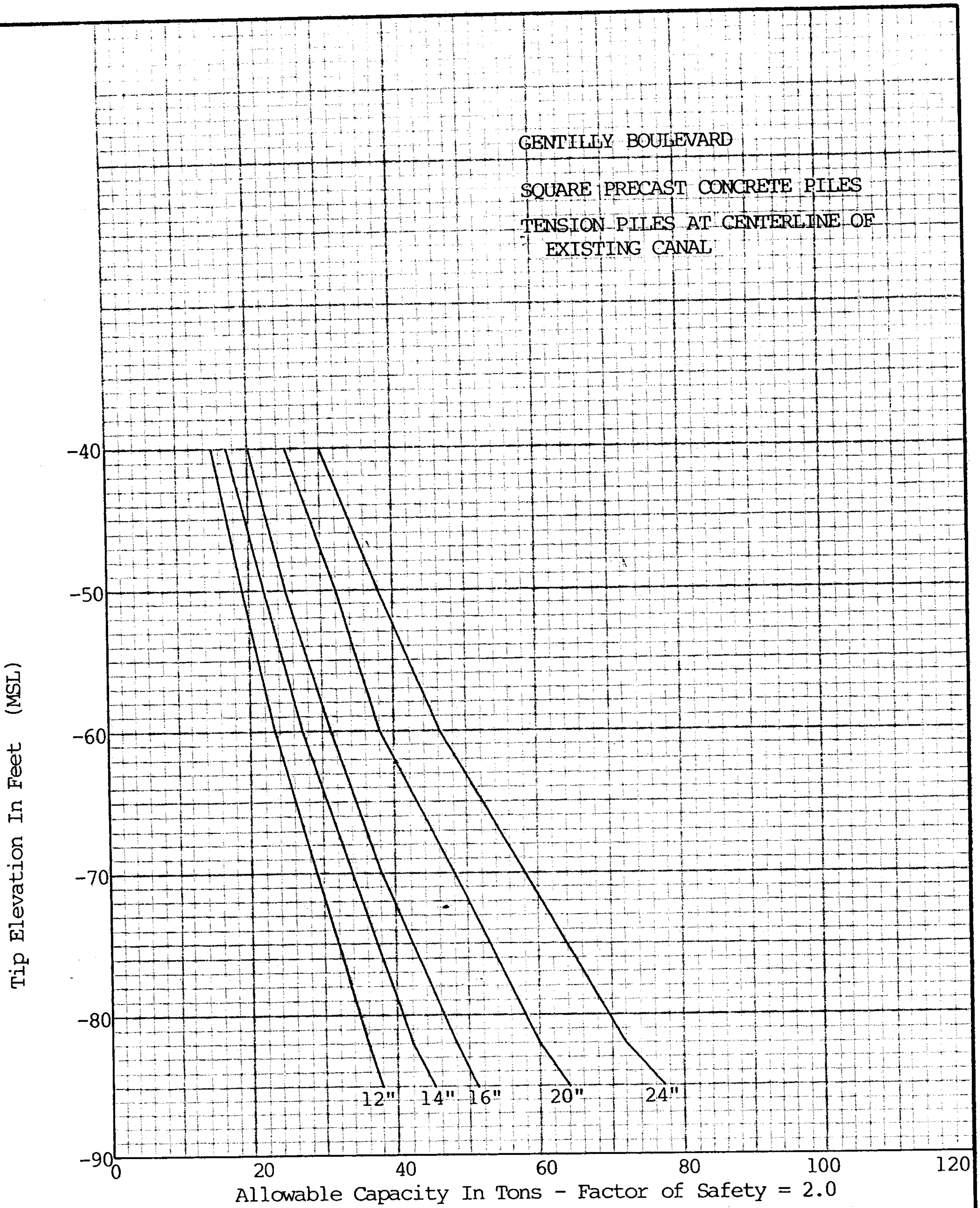

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Fig. 17

EUSTIS ENGINEERING COMPANY
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METAIRIE, LA.

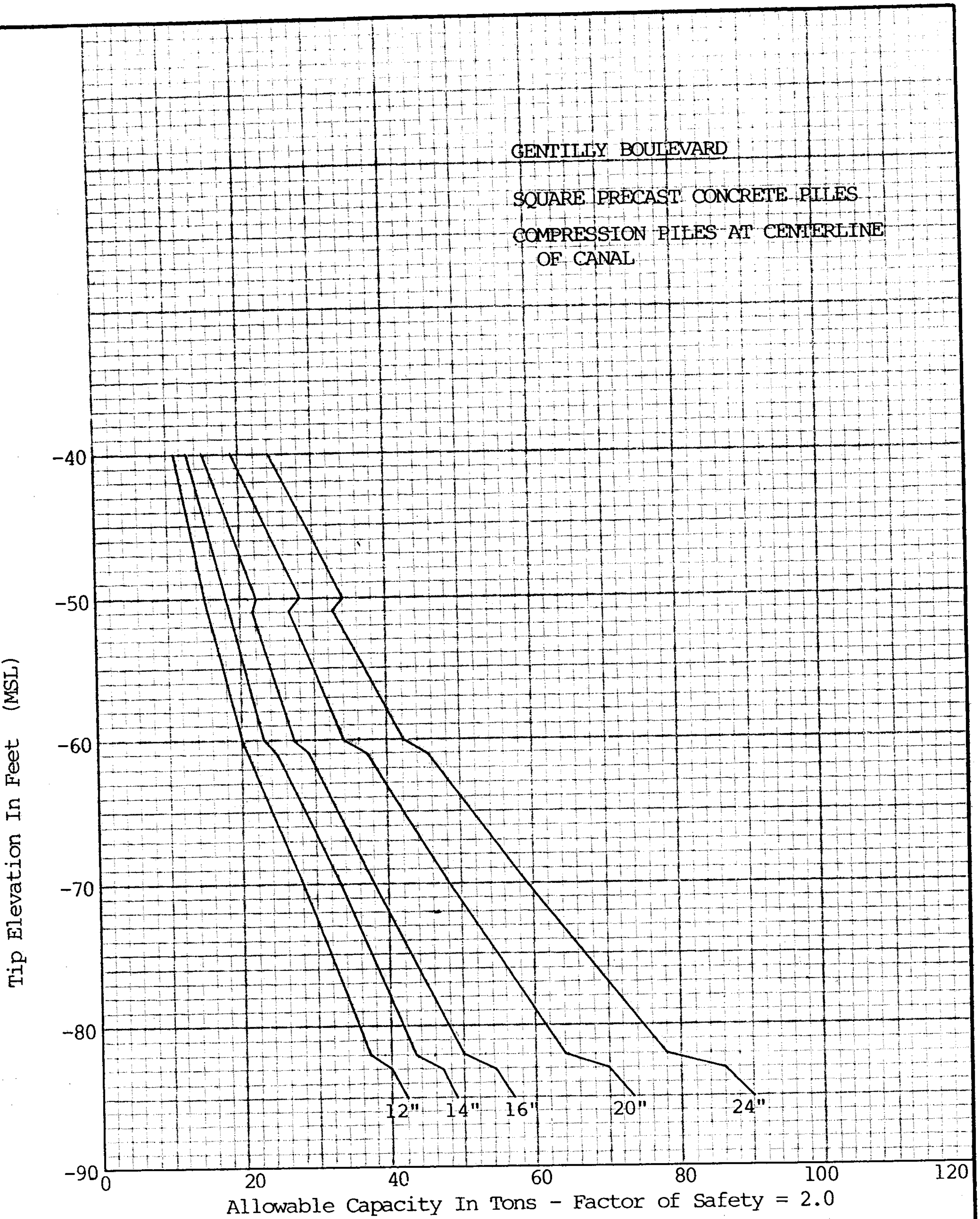

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Fig. 18

EUSTIS ENGINEERING COMPANY
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METAIRIE, LA.

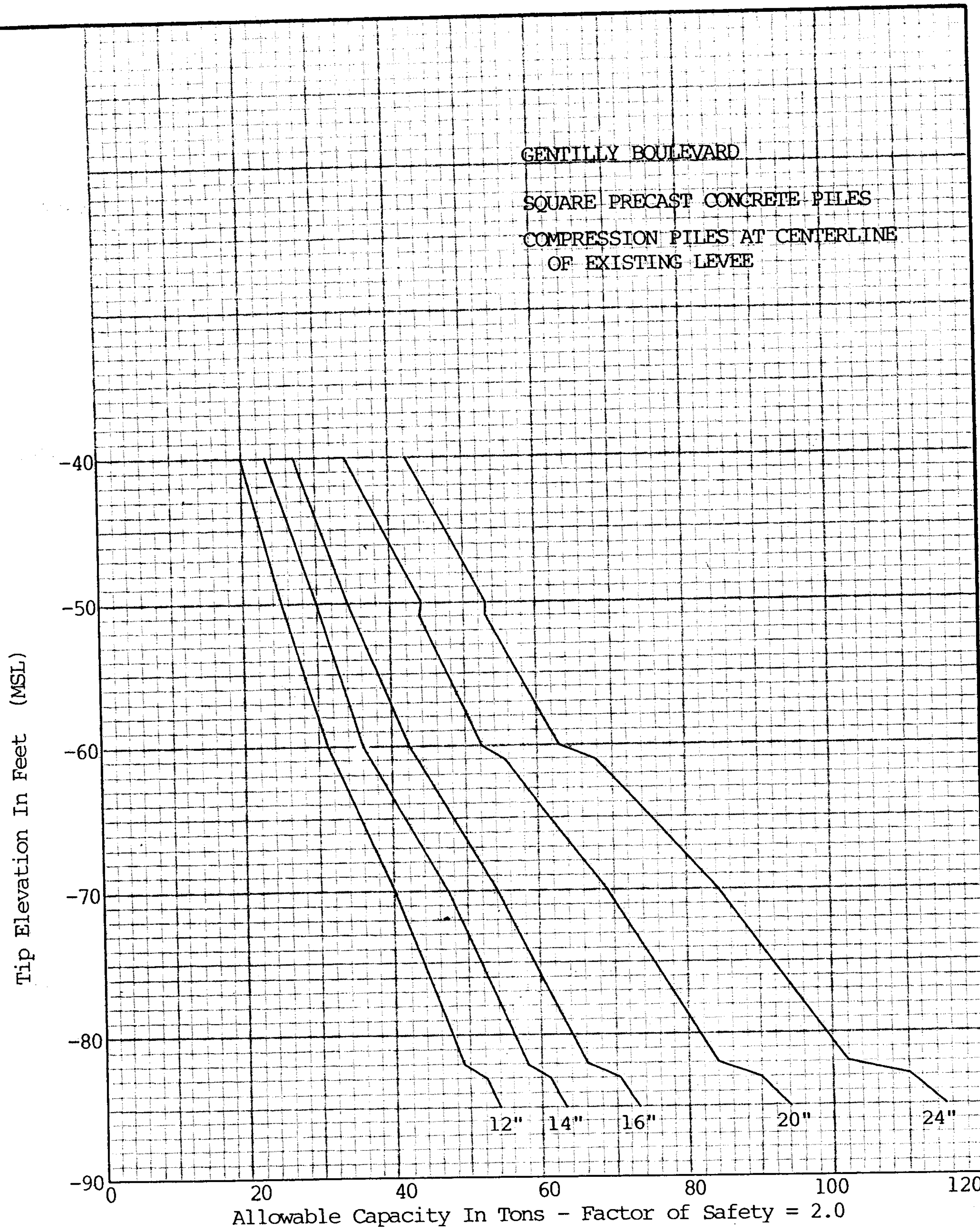

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Fig. 19

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA

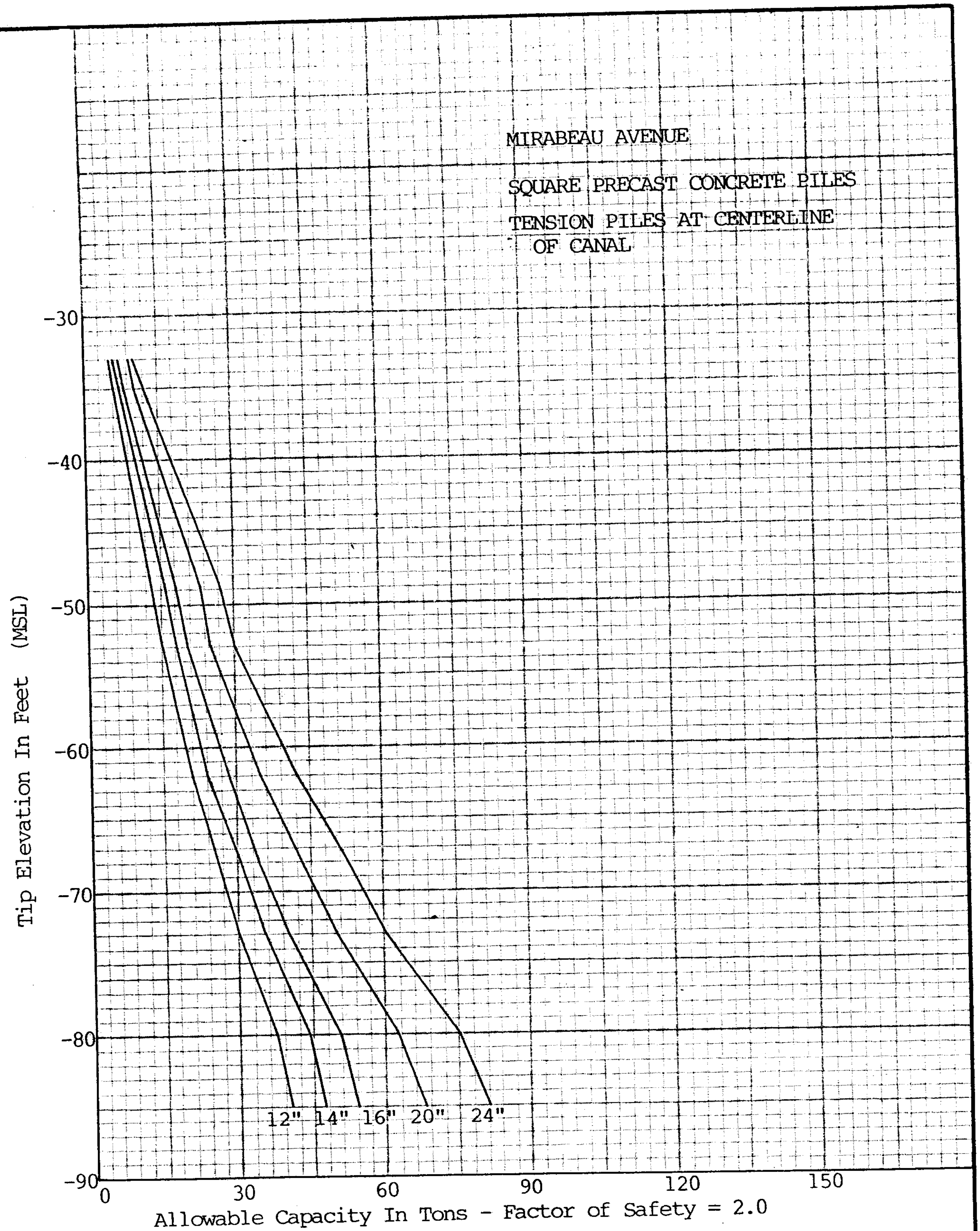

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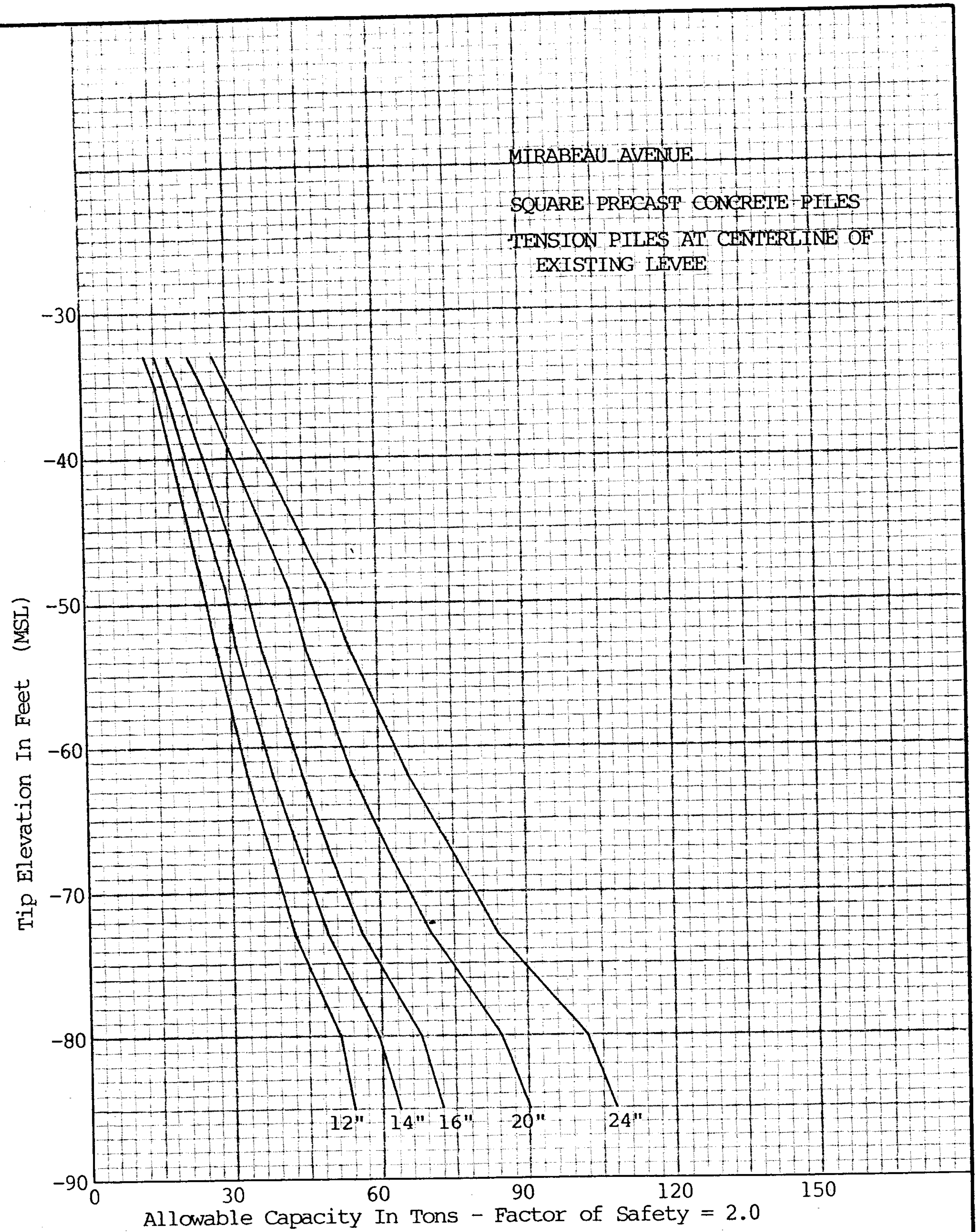
Fig. 20

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

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Fig. 21

EUSTIS ENGINEERING COMPANY
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METAIRIE, LA.

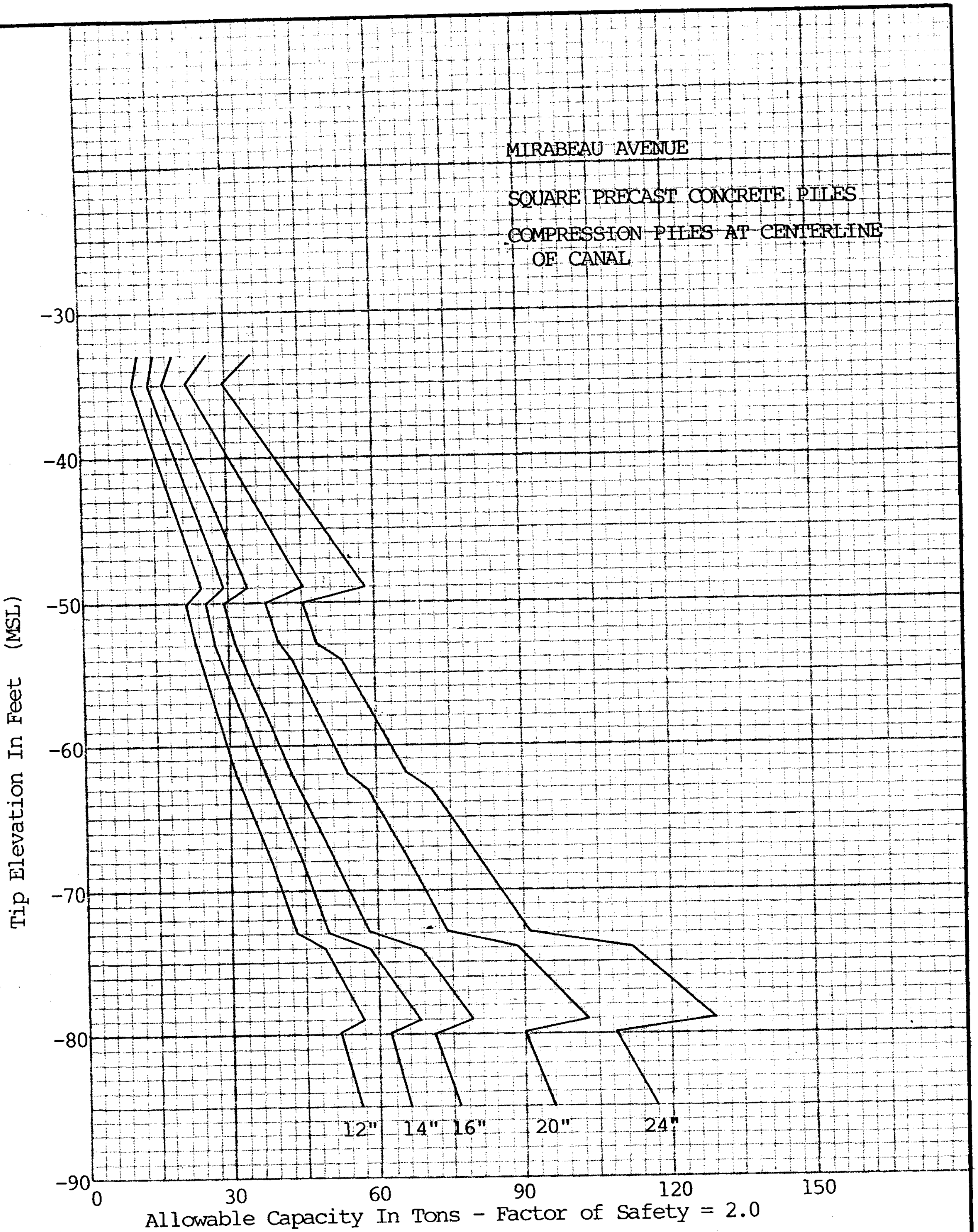

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Fig. 22

EUSTIS ENGINEERING COMPANY
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METAIRIE, LA.

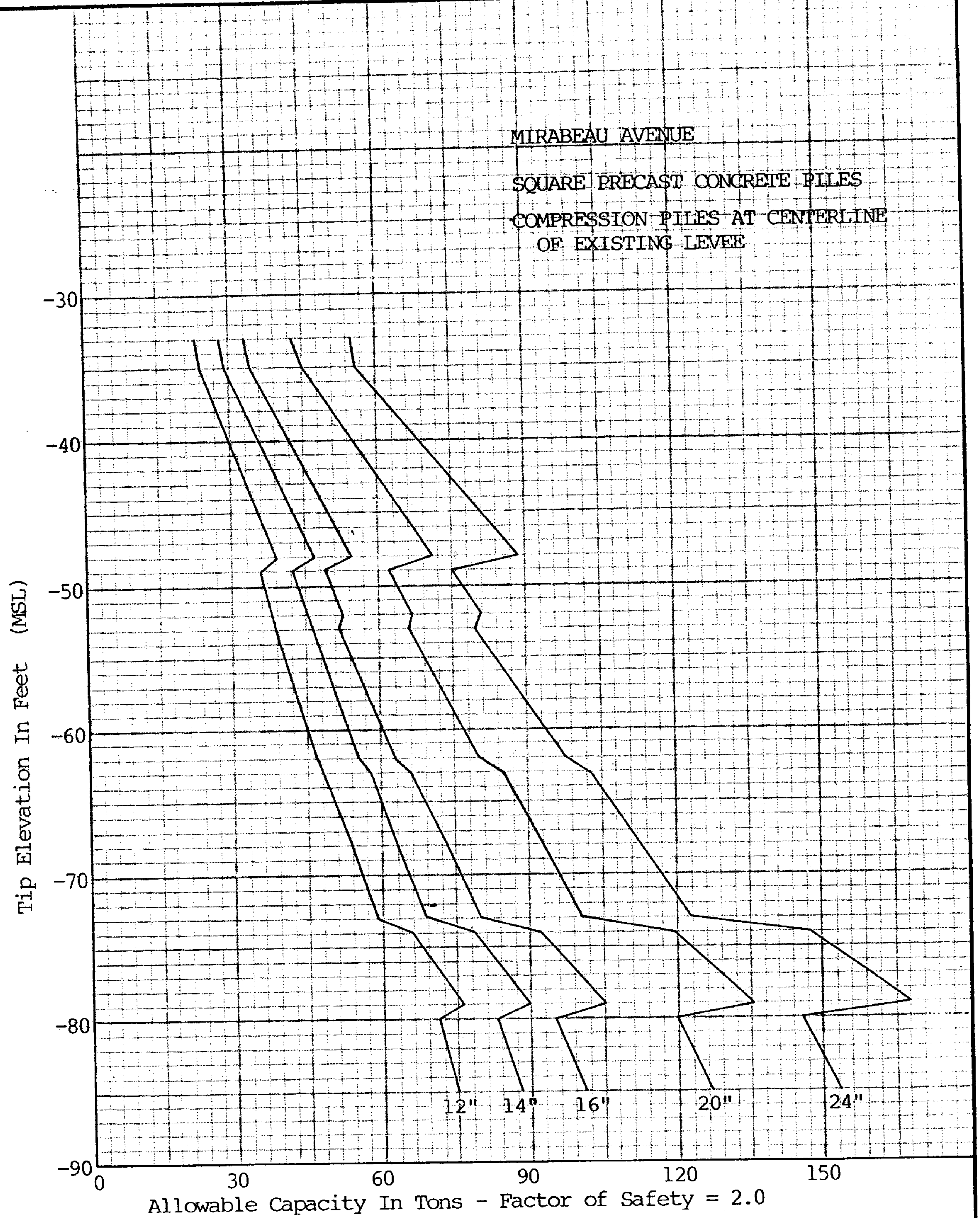

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Fig. 23

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

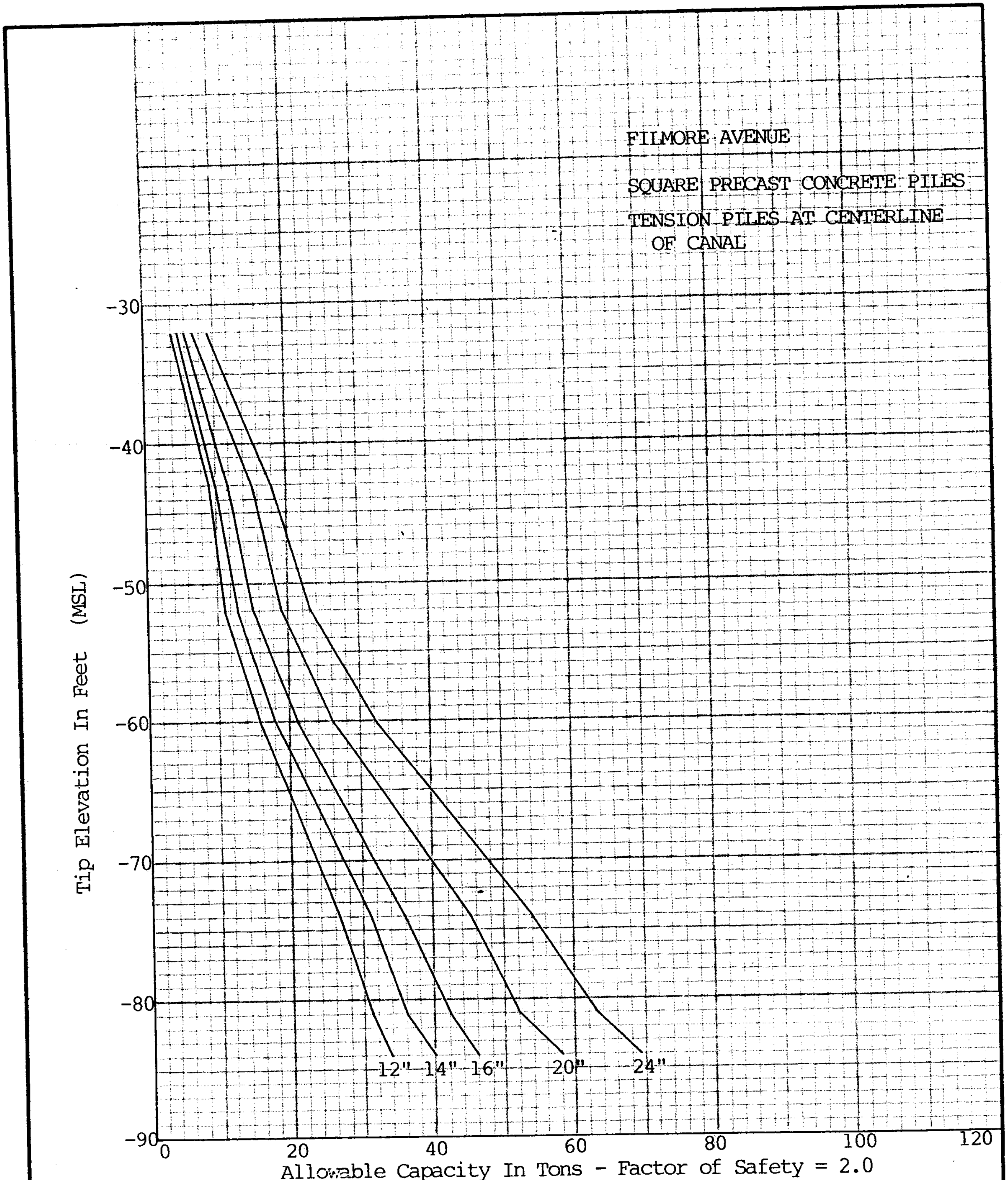

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Fig. 24

EUSTIS ENGINEERING COMPANY
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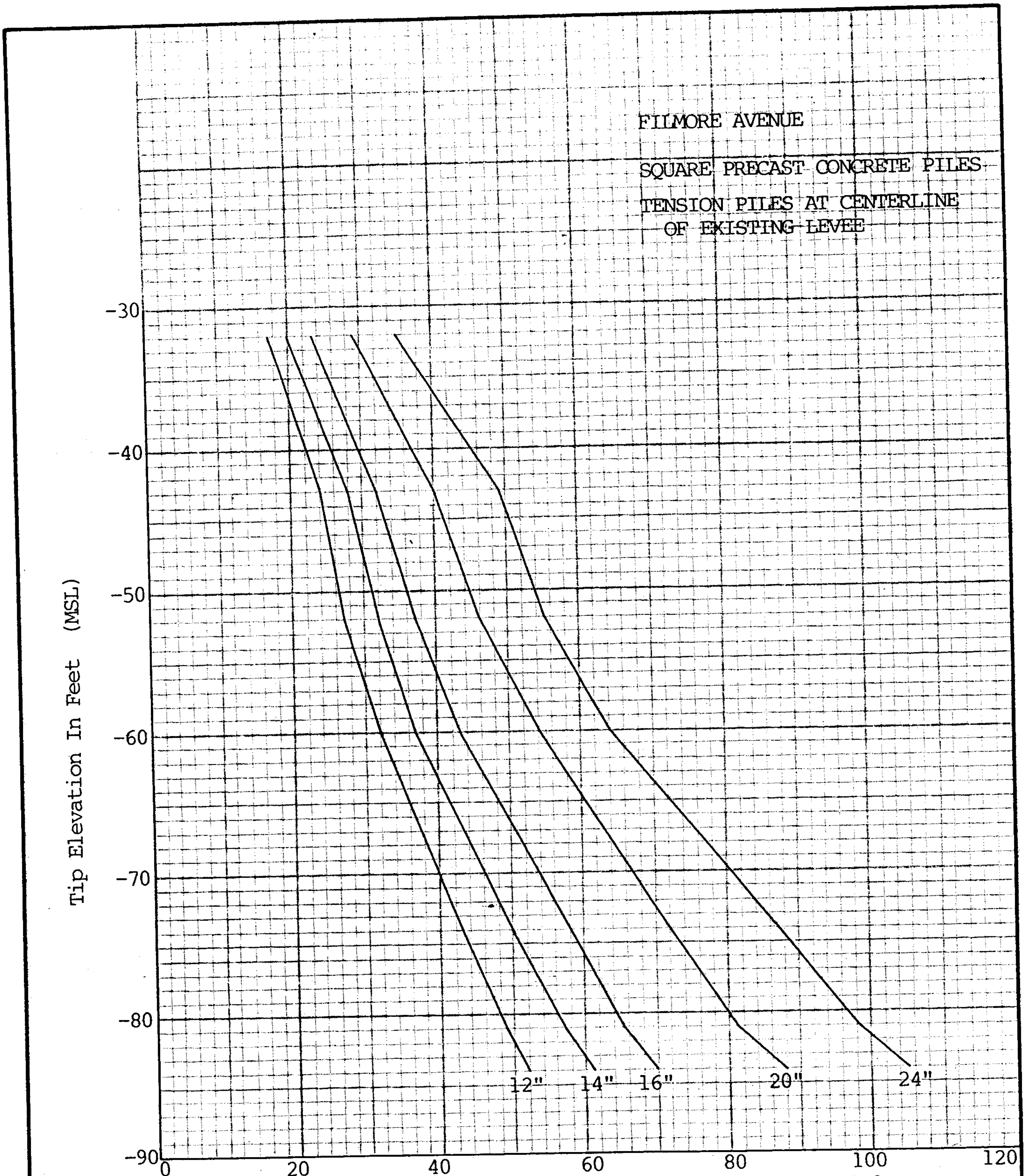
METAIRIE, LA.



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New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
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Fig. 25

EUSTIS ENGINEERING COMPANY
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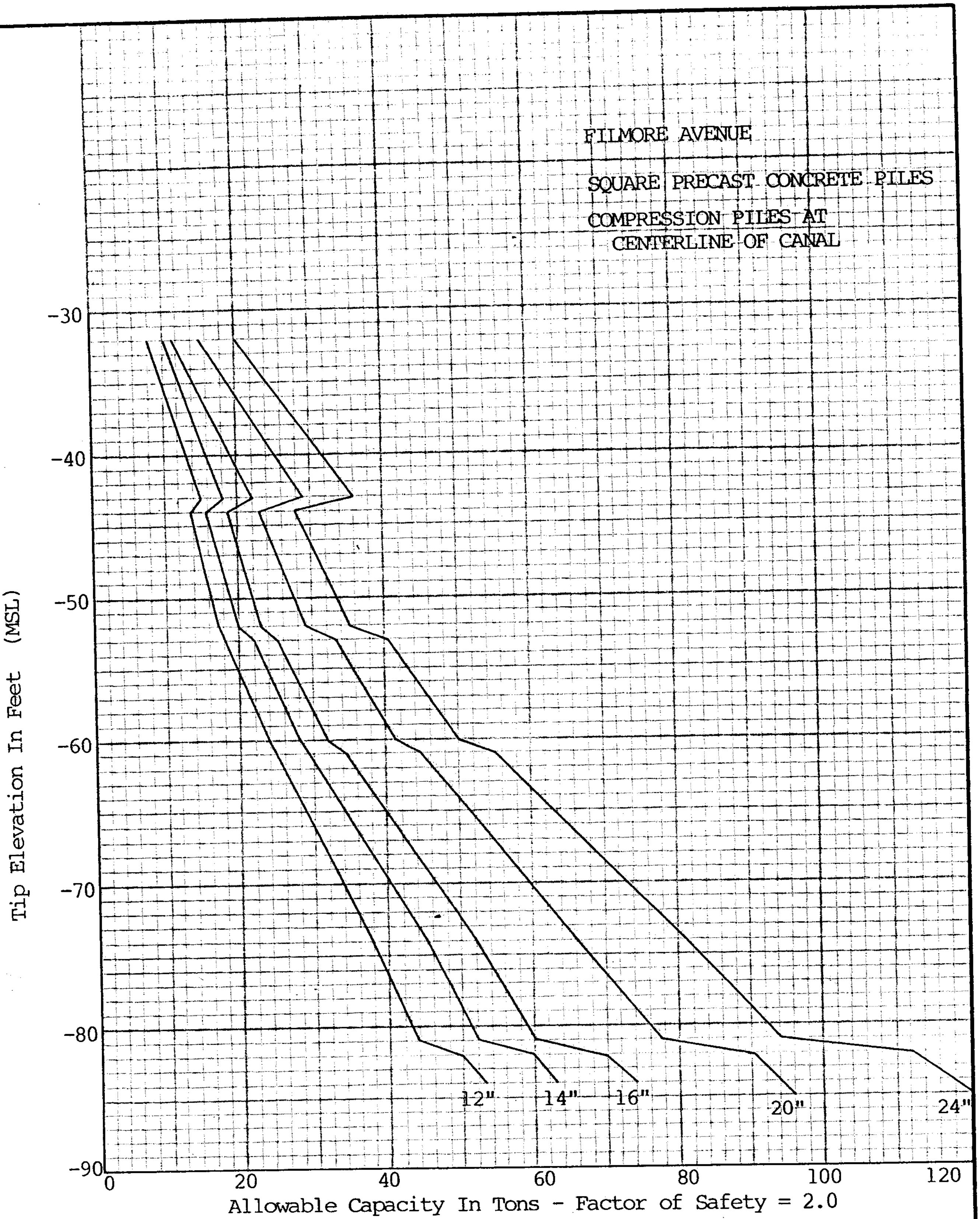
METAIRIE, LA.



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Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
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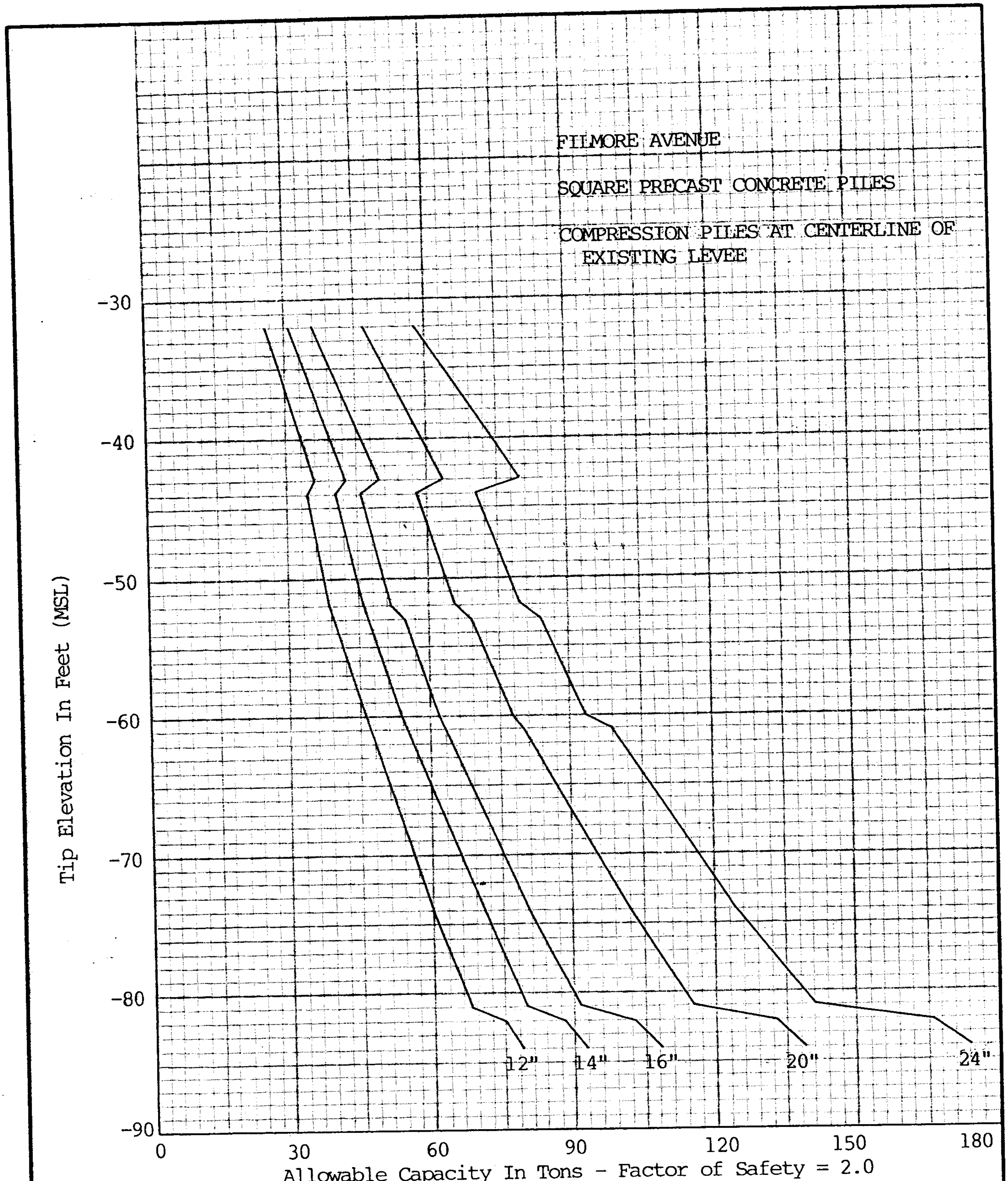
Fig. 26



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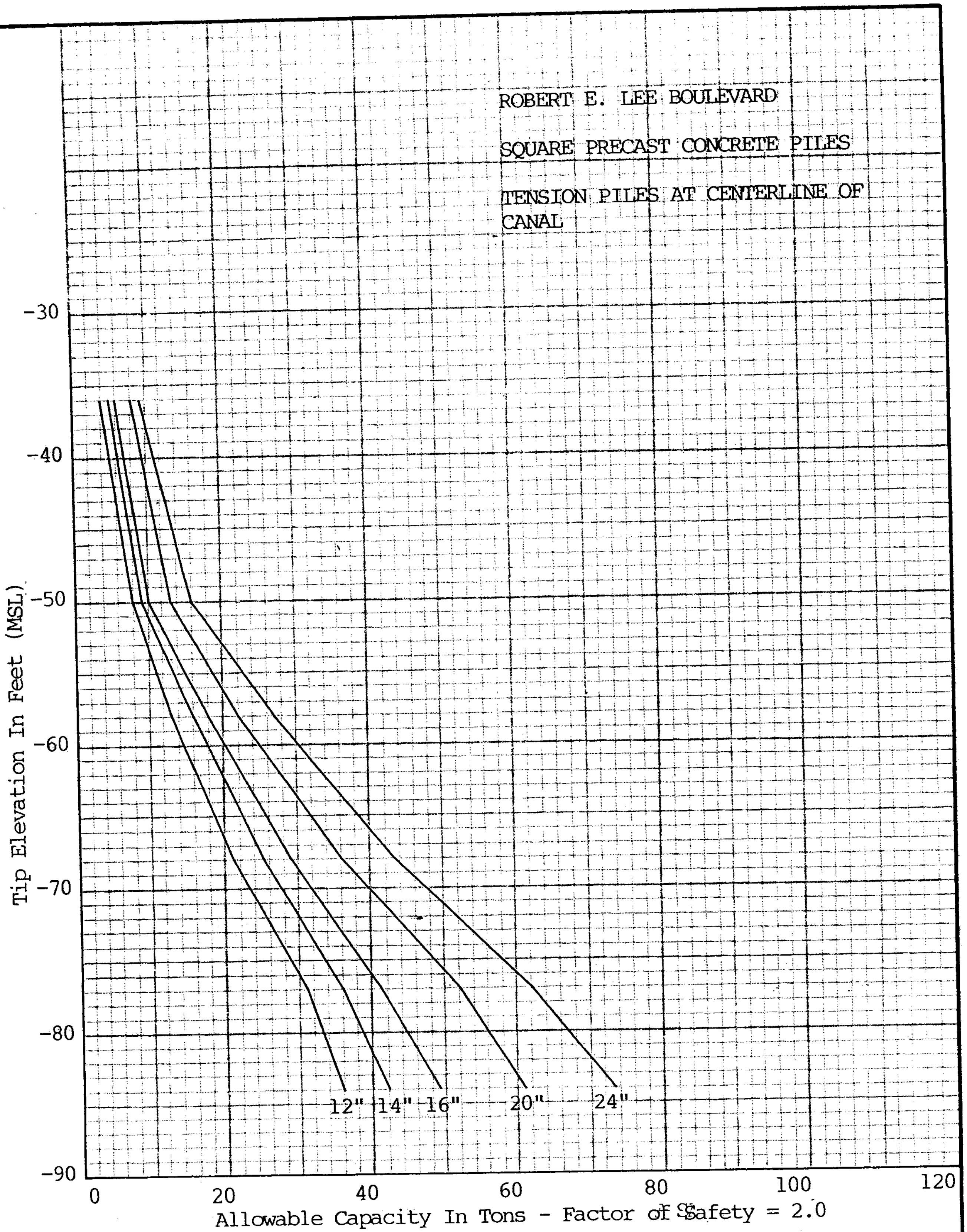
For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 27

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

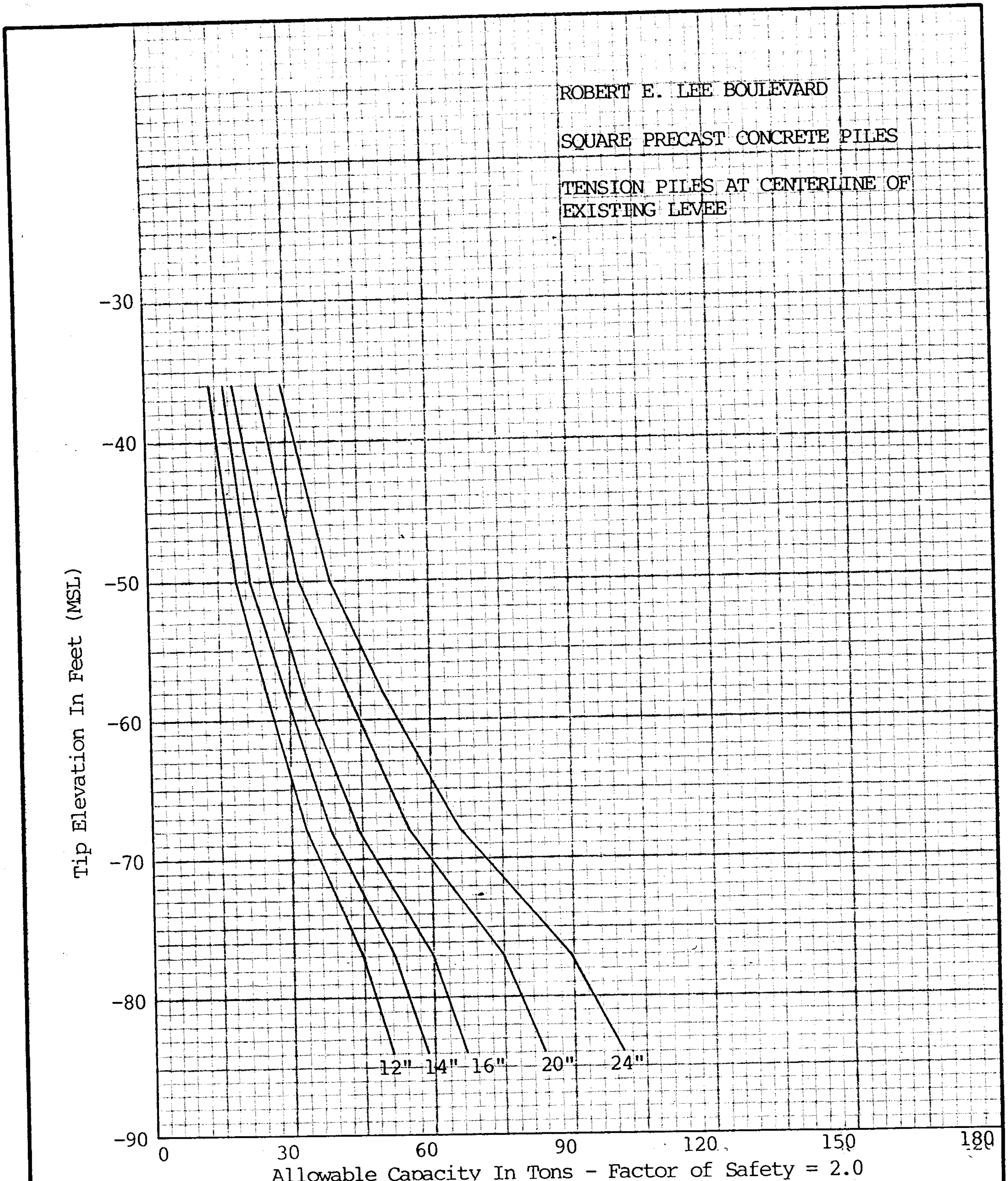


Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., New Orleans, Louisiana

Fig. 28



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New Orleans, Louisiana
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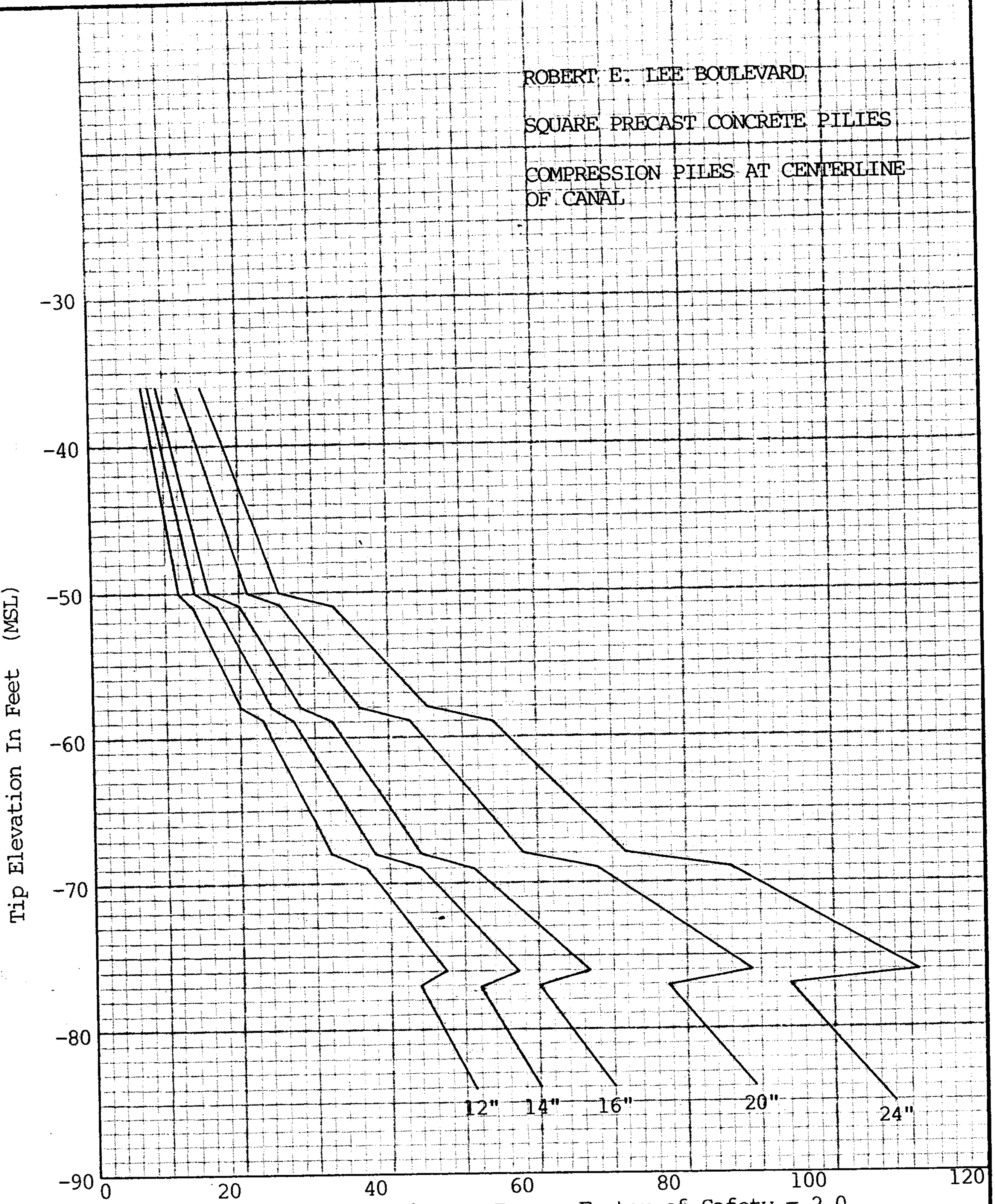
Fig. 29

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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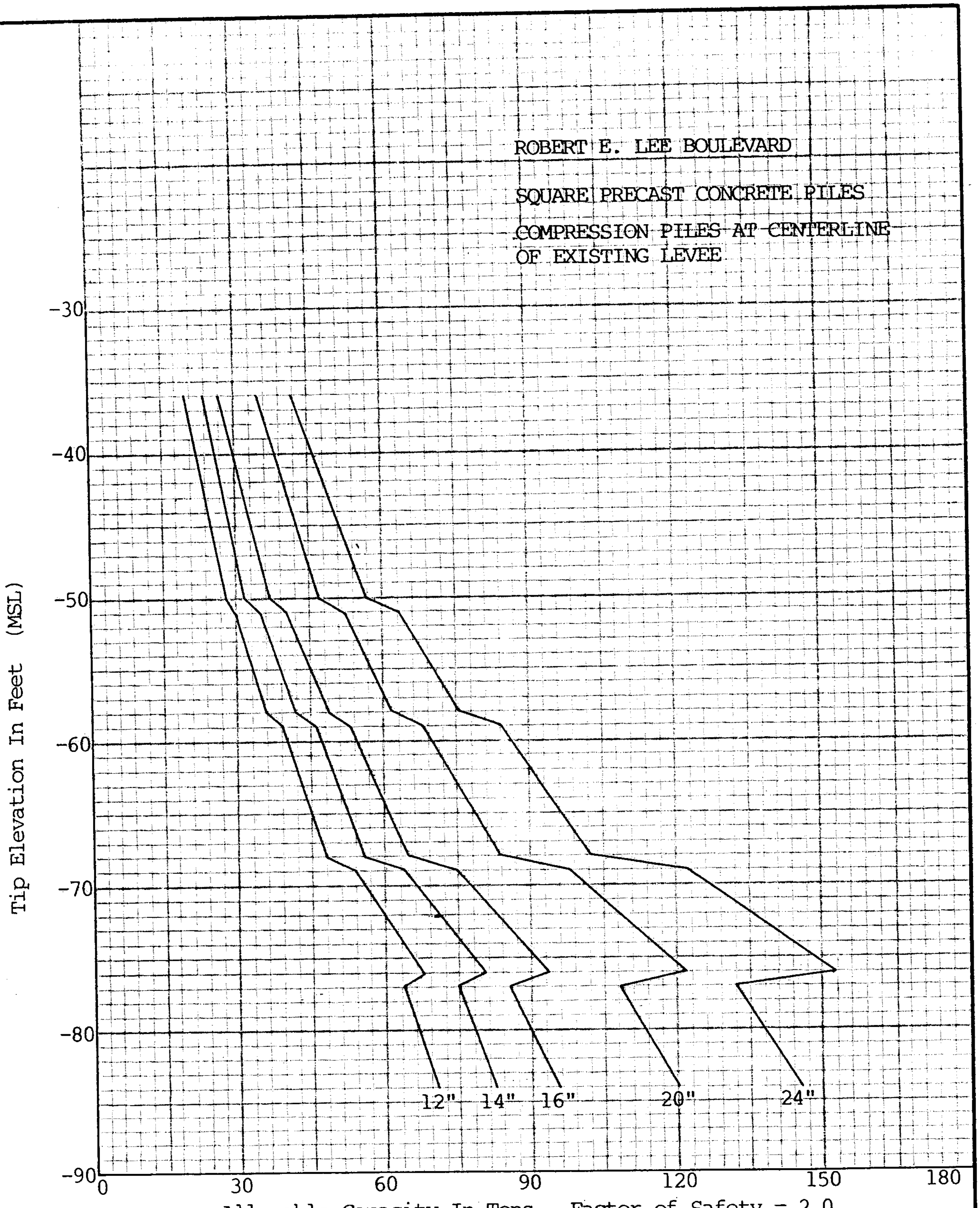
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 30

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Allowable Capacity In Tons - Factor of Safety = 2.0

Geotechnical Investigation
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Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
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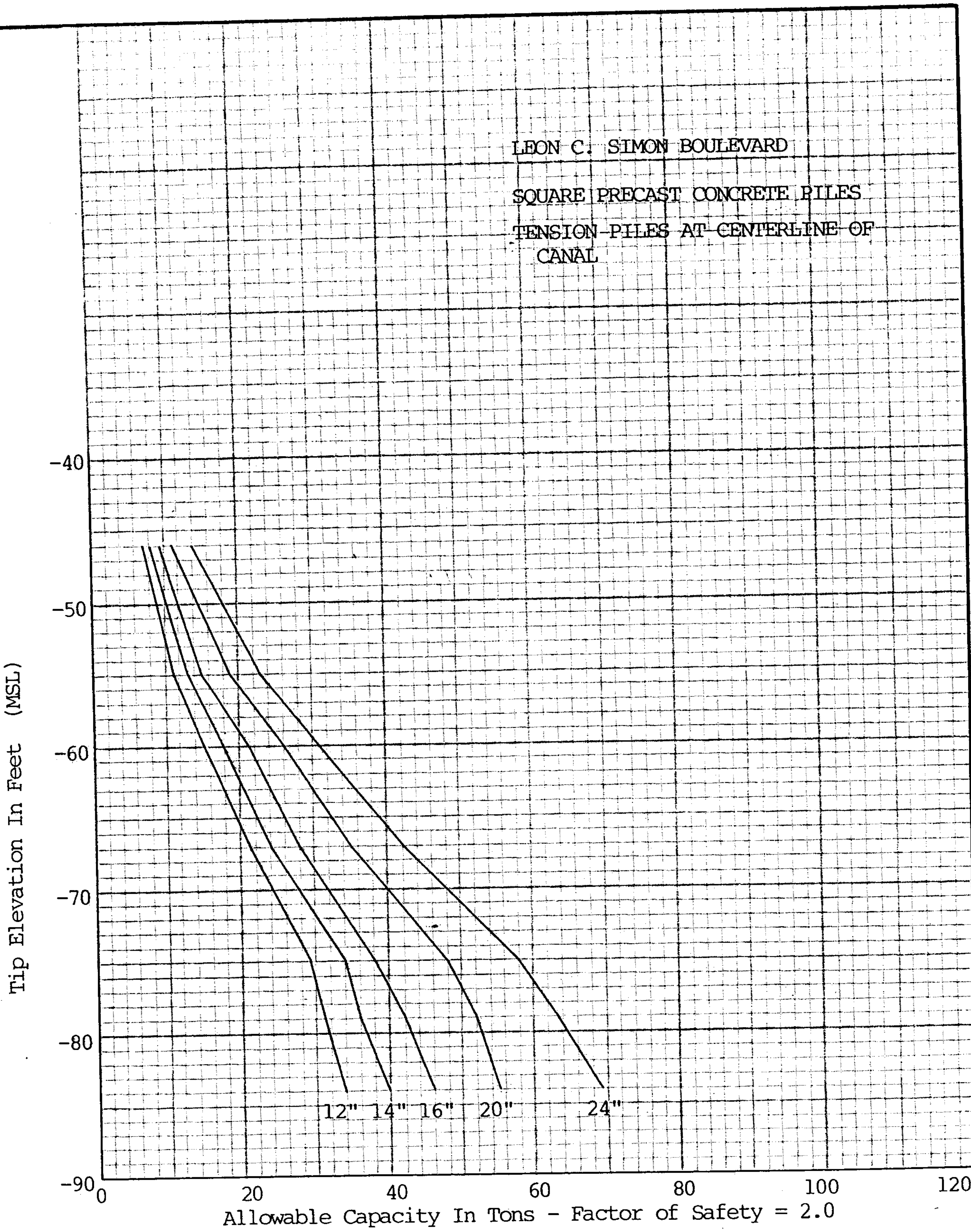
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 31

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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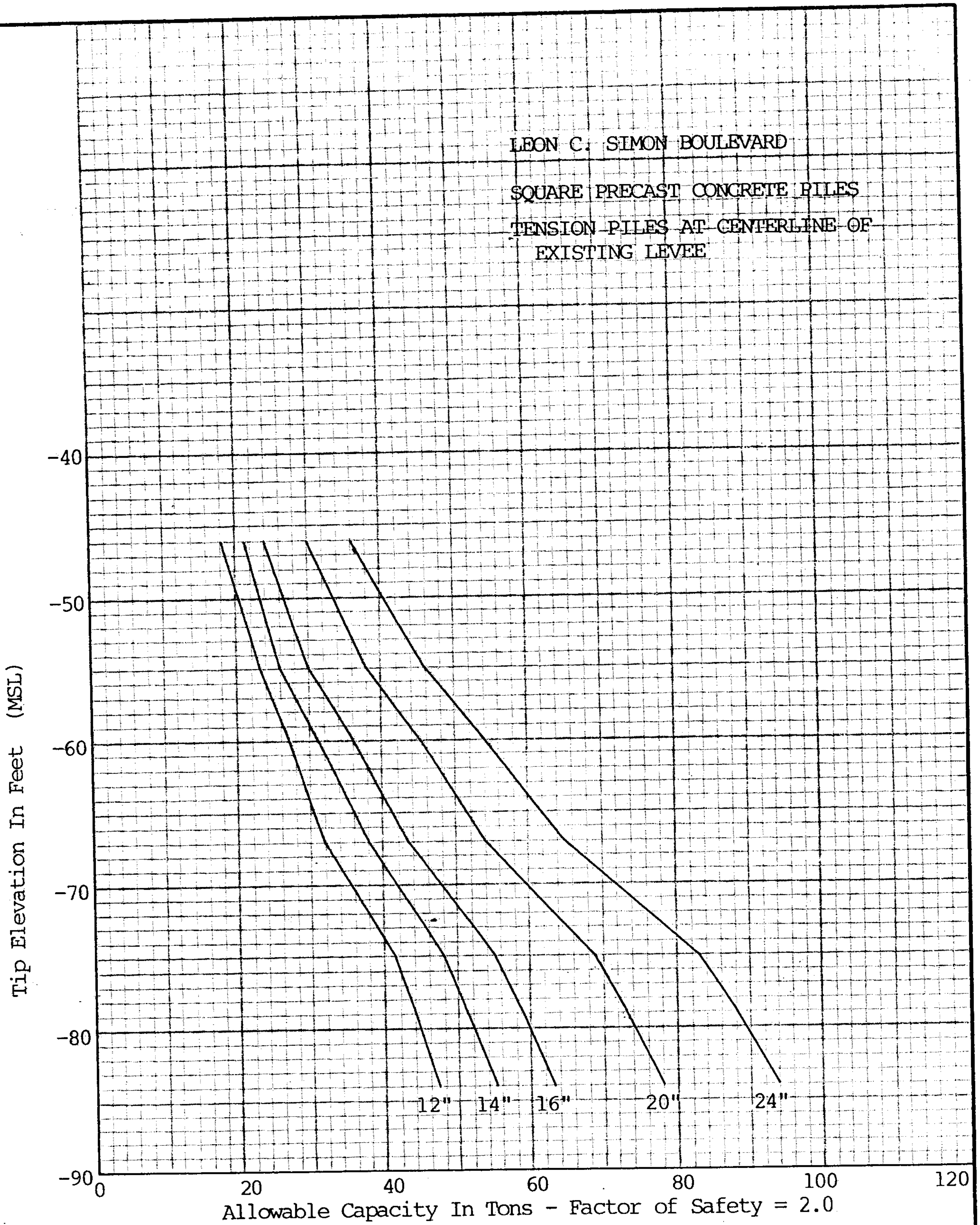
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 32

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Geotechnical Investigation
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Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
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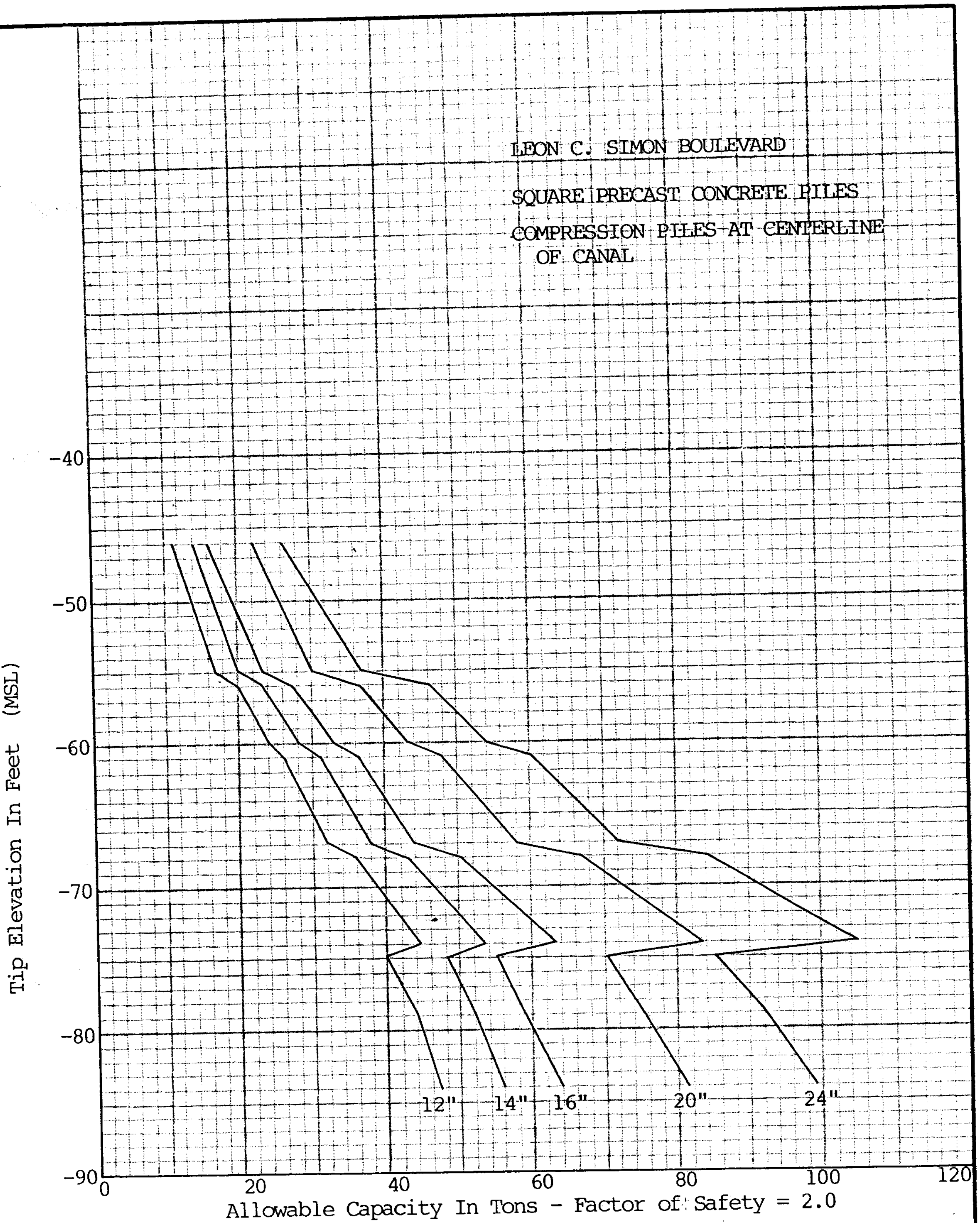
Fig. 33

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



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New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
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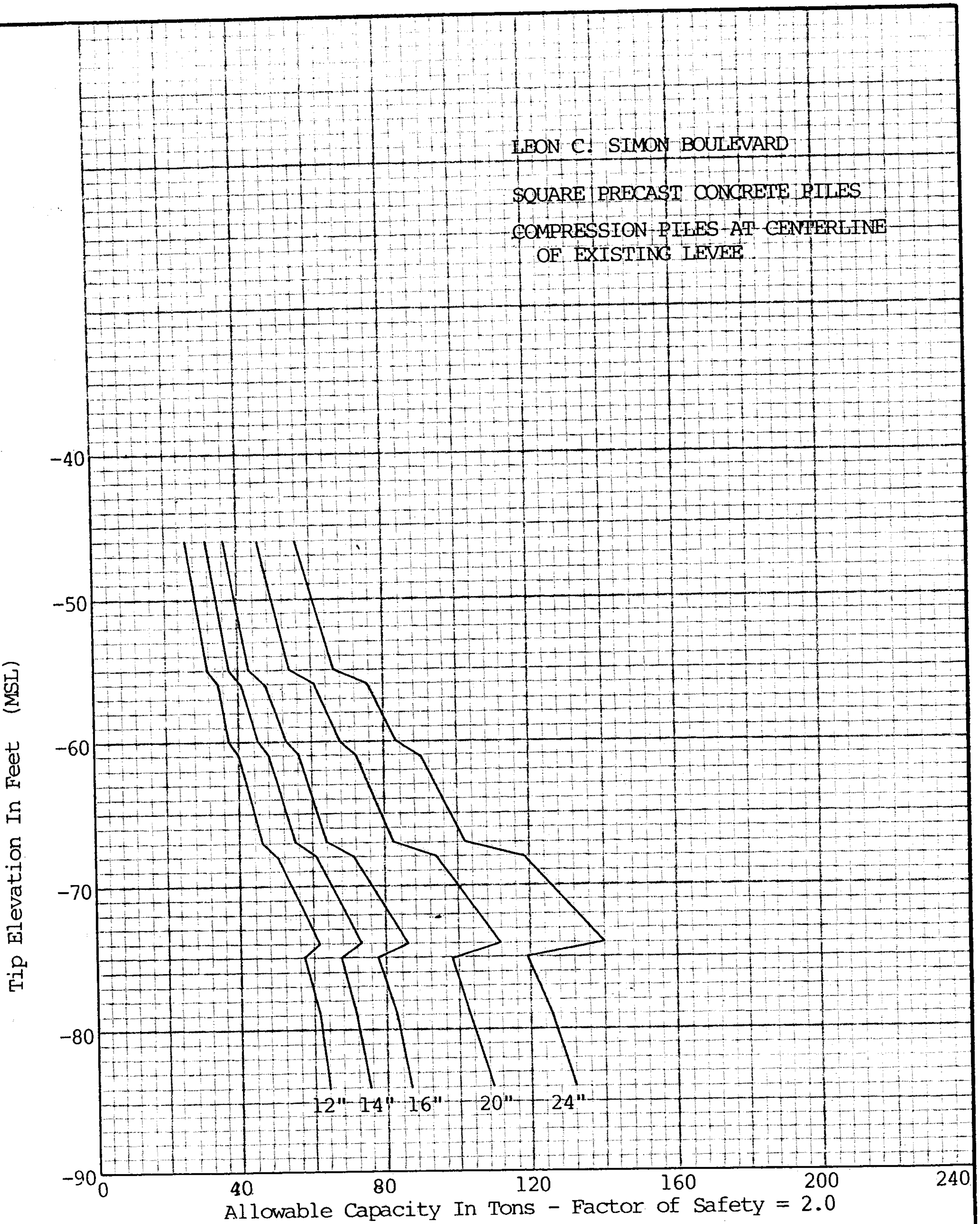
Fig. 34

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Geotechnical Investigation
London Avenue Canal
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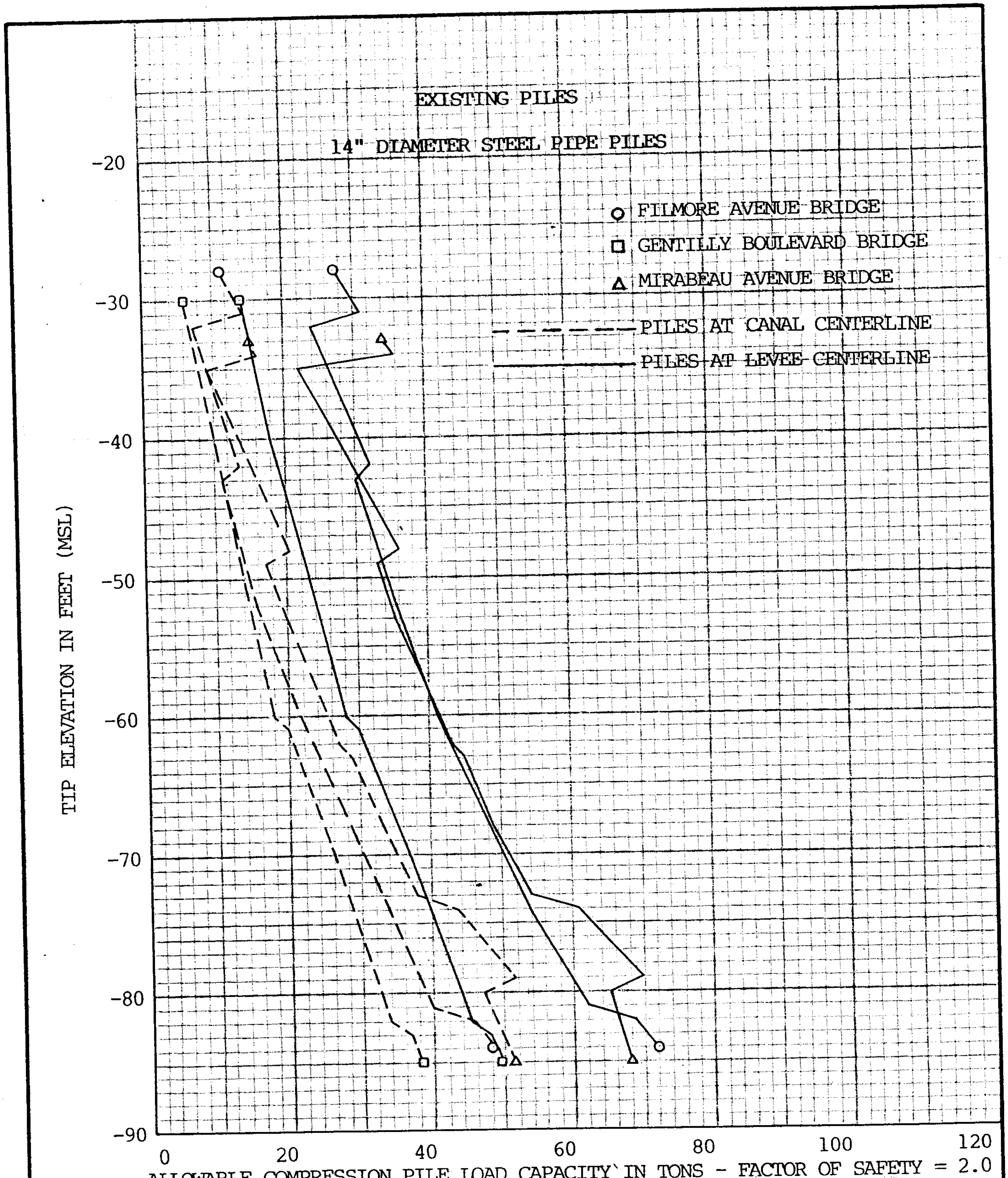
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 35

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Geotechnical Investigation
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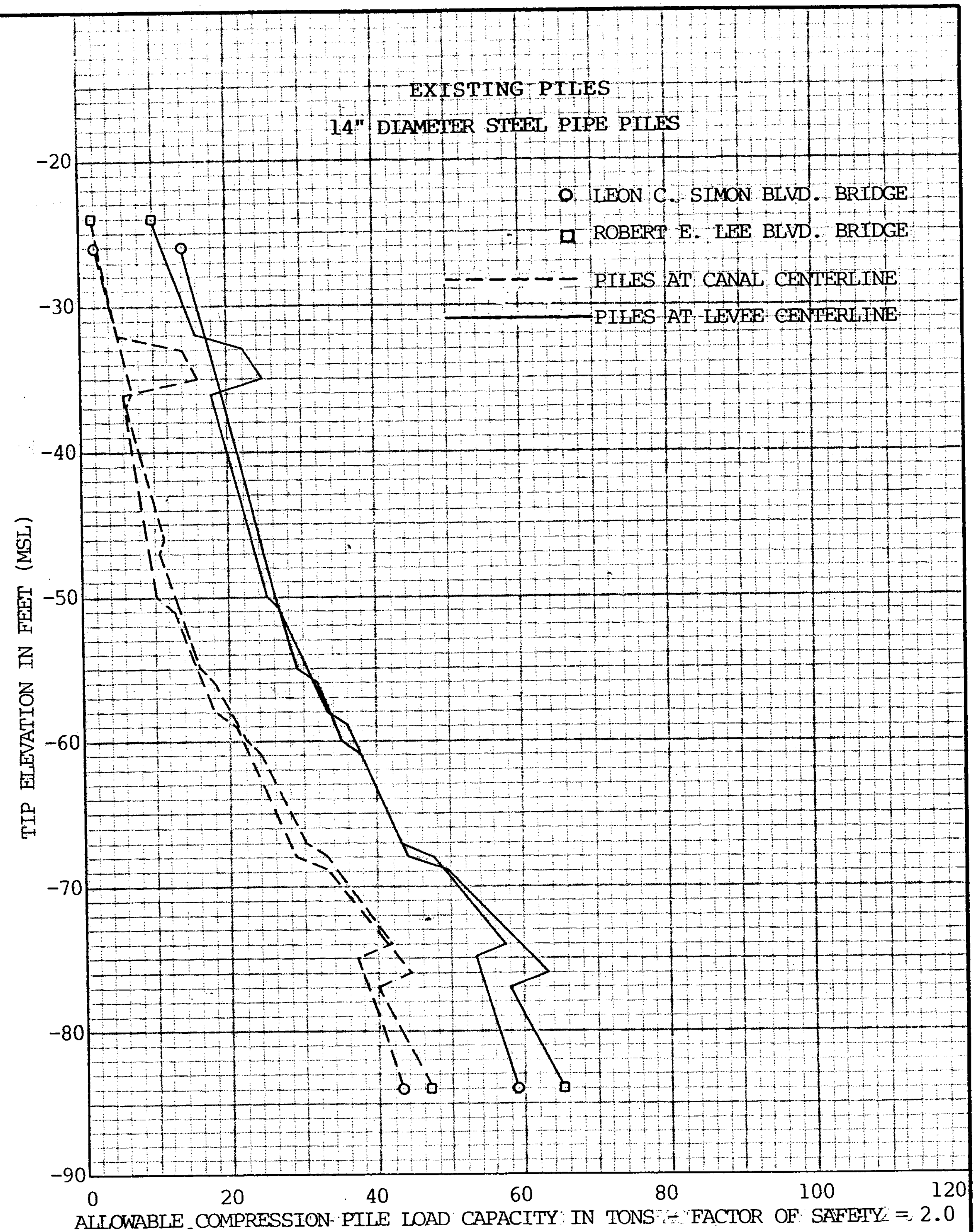
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 36

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Geotechnical Investigation
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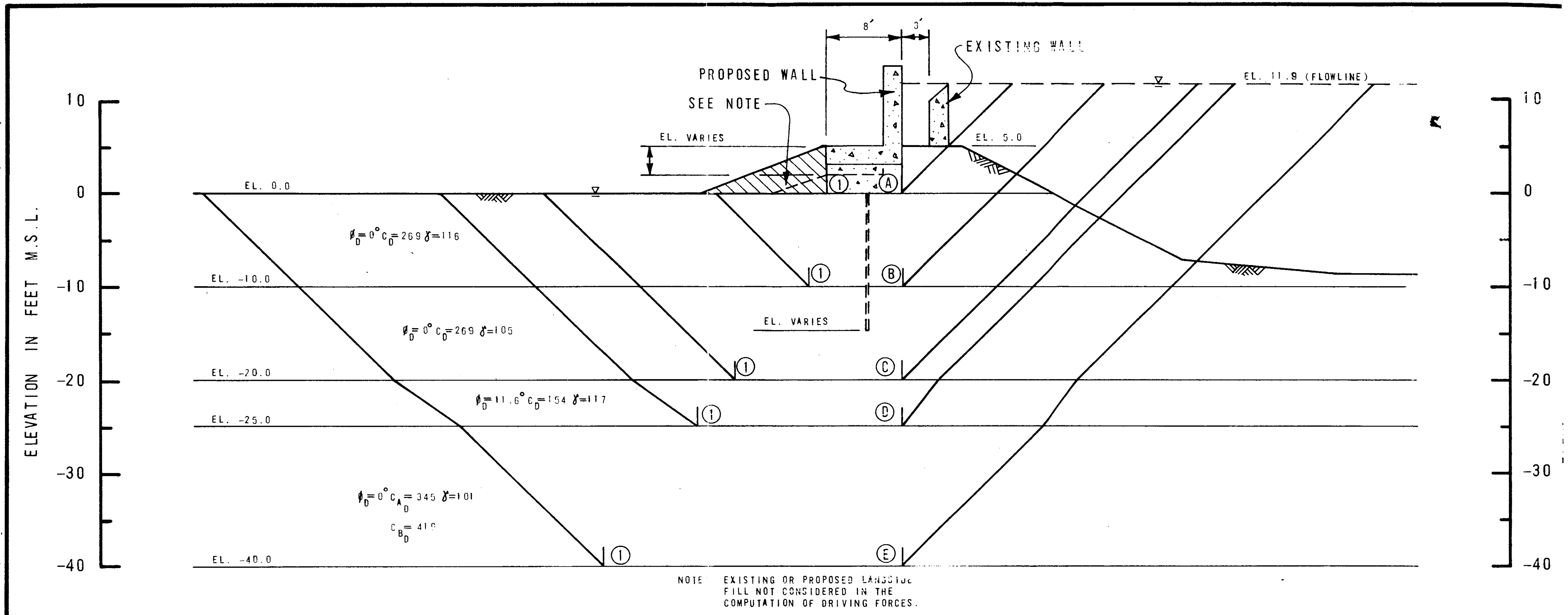
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana
Burk & Associates, Inc., New Orleans, Louisiana

Fig. 37

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



SLIP SURFACE			DRIVING FORCE			RESISTING FORCE				ΔE_C
NUMBER		ELEV.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	
(A)	(1)	0	4880	0	4880	5725	0	0	5725	-845
(B)	(1)	-10	20106	5800	14306	7960	2690	5380	16030	-1724
(C)	(1)	-20	44579	22649	21930	10047	4842	10760	25649	-3719
(D)	(1)	-25	60577	35161	25416	13501	8546	15407	37448	-12032
(E)	(1)	-40	120662	88446	32216	20392	13406	25757	59557	-27341

NOTE: SEE FIGURE 6 FOR FLOODSIDE STABILITY ANALYSES.

LEGEND

γ=SATURATED UNIT WEIGHT IN POUNDS PER CU. FT.
C=COHESION IN POUNDS PER SQ. FT.
C_A=AVERAGE COHESION IN POUNDS PER SQ. FT. IN VERTICAL REACH OF STRATUM.
C_B=COHESION IN POUNDS PER SQ. FT. ALONG HORIZONTAL PLANE AT BOTTOM OF STRATUM.
φ=ANGLE OF INTERNAL FRICTION IN DEGREES.
SUBSCRIPT D REFERS TO DEVELOPED SHEAR STRENGTH PARAMETER.
R=HORIZONTAL FORCE DUE TO SOIL SHEAR STRENGTH.
D=HORIZONTAL COMPONENT OF SOIL WEIGHT IN WEDGE.
SUBSCRIPT A REFERS TO ACTIVE WEDGE.
SUBSCRIPT B REFERS TO CENTRAL BLOCK.
SUBSCRIPT P REFERS TO PASSIVE WEDGE.
▽ PIEZONATRIC SURFACE
STABILITY BASED ON LMVD METHOD OF PLANES.
ΔE_C=SUMMATION OF HORIZONTAL FORCES.
$\Delta E_C = \Sigma D - \Sigma R$

GEOTECHNICAL INVESTIGATION
LONDON AVENUS CANAL
LEVEE AND FLOODWALL IMPROVEMENTS
ORLEANS LEVEE BOARD PROJECT NO. 2049-0269
NEW ORLEANS, LOUISIANA

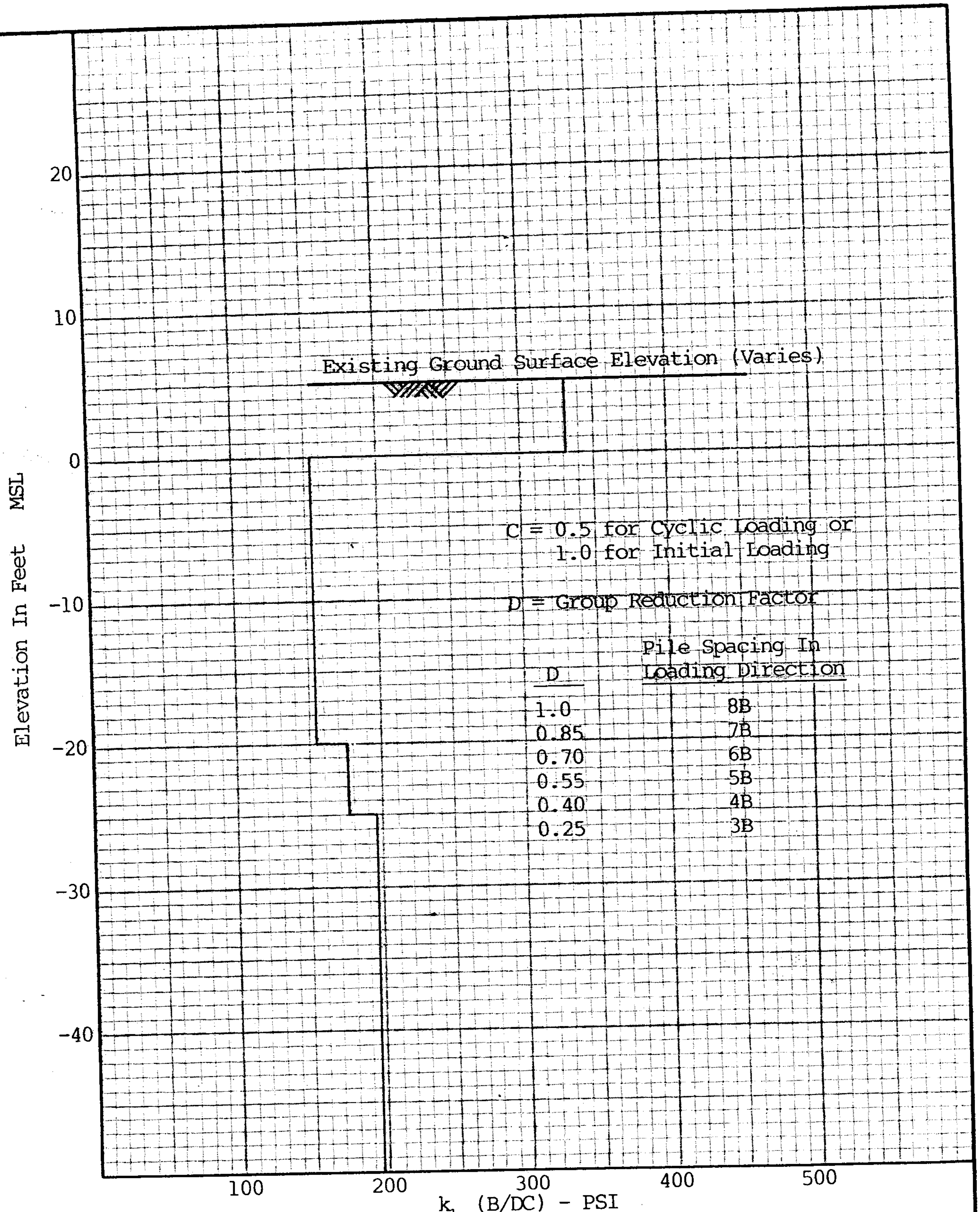
DEEP SEATED STABILITY ANALYSIS

FOR
THE BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
NEW ORLEANS, LOUISIANA

BURK & ASSOCIATES, INC.
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
MARCH 1986 METAIRIE, LA.

FIGURE 38



Reach I - Station 0+00 To 21+00
Modulus of Horizontal Subgrade Reaction For Laterally Loaded Piles

Geotechnical Investigation
London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., New Orleans, Louisiana Fig. 39

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

CAPACITY OF PILE GROUPS

$$Q_a = \frac{P \times L \times c}{(FSF)} + \frac{2.6\, q_u \left(1 + 0.2\, \frac{w}{b}\right) A}{(FSB)}$$

In Which:

Q_a = Allowable load carrying capacity of pile group, lb

P = Perimeter distance of pile group, ft

L = Length of pile, ft

c = Average (weighted) cohesion or shear strength of material between surface and depth of pile tip, psf (c = one-half the unconfined compressive strength)

q_u = Average unconfined compressive strength of material in the zone immediately below pile tips, psf

w = Width of base of pile group, ft

b = Length of base of pile group, ft

A = Base area of pile group, sq ft

(FSF) = Factor of safety for the friction area = 2

(FSB) = Factor of safety for the base area = 3

The values of c and q_u used in this formula should be based on applicable soil data shown on the Summary of Laboratory Test Results tabulations and logs of soil borings for this report. In the application of this formula, the weight of the piles, pile caps and mats, considering the effect of buoyancy, should be included.

SPACING OF PILE GROUPS

$$SPAC = 0.05\,(L_1) + 0.025\,(L_2) + 0.0125\,(L_3)$$

In Which:

SPAC = Center to center of piles, ft

L_1 = Pile penetration up to 100 feet

L_2 = Pile penetration from 101 to 200 feet

L_3 = Pile penetration beyond 200 feet

Note: Minimum pile spacing = 3 pile diameters (center to center)

Fig. 40