A0006787

CELMN-ED-SP (CELMV-ED-SP/15 May 90) (1110-2-1150a) 2d Ind
Mr. Stutts/kn/2614
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal

DA, New Orleans District, Corps of Engineers, P. O. Box 60267, New Orleans, LA 70160-0267 22 Oct 90

FOR Commander, Lower Mississippi Valley Division, ATTN: CELMV-ED-PG

The following are our responses to comments contained in the 1st Endorsement.

a. Comment a. Para. 7c. Concur. The per annum interest rate is 3.125 percent.

b. Comment b. Para. 14 and Appendix DD, page DD-9. The design computations for the Veterans Highway Bridge are adequate to develop the GDM scope cost estimate. During preparation of plans and specifications we will prepare a more punctilious design that addresses all conventional loading including hydraulic loads (horizontal and uplift) as recommended by American Association of State Highway and Transportation Officials (AASHTO). In addition, CELMV's recommendation on establishing pile fixity is noted and we will comply in the detailed design phase.

c. Comment c. Para. 29b. Do not concur. Reference CEMRC-ED-GS memorandum dated 24 July 89, para. 3. A penetration to head ratio of 2.5 to 3:1 is recommended in the referenced memorandum. For certain projects a penetration to head ratio of less than 2.5 was authorized. The factors stated in the memorandum which cause the tip to be arbitrarily increased by a penetration to head ratio are unknown variations in ground surface elevations and soil conditions. The 2.5:1 penetration to head ratio was used because of the following:

(1) The ground surface elevations are based on surveys at 100 ft. intervals.

(2) Two surveys along the canal were done in the last 10 years.

(3) The velocities in the canal are too low to cause scour.

1 John G.

(4) Borings were taken at 350 ft. intervals by the A/E on both sides of the canal and were supplemented by USACE check borings.

(5) The existing levee is over 30 years old.

All hurricane protection in urban areas is critical in nature; however, no other hurricane protection project has had the level of borings or surveys as the 17th St. Canal project. The 3 to 1 minimum ratio was used on other New Orleans projects because of the CEMRC-ED-GS memorandum dated 23 Dec 87 which required the 3 to 1 ratio. No GDM has been submitted for a 3 to 1 ratio in an existing levee since the July 89 criteria.

The sheetpile sections on plates 101 through 103 and 111 have penetration ratios of 2.5 to 1 and an S-CASE F.S. of 1.2 for canal water level of 0.0. Sections on plates 104 through 105 and 112 have tip elevations deeper than required for an S-CASE F.S. = 1.5 or 3:1 ratio for the bulkhead case. The sheetpile sections on plates 110 and 113 have an S-CASE F.S. of 1.4 for canal water level of 0.0 (see Encl 2). Additionally, the existing sheetpile wall has served as a permanent bulkhead retaining as much as 4 ft. of soil for at least 18 years (Orleans Levee Board 1971 Surveys). We will monitor the sheetpile wall being constructed by the local interests on the Orleans side of the canal. We will consider driving the sheetpile deeper instead of cutting the sheetpile in 1994 during capping.

d. Comment d, Para. 29c. The analyses presented were for the S-CASE soil strengths, a canal level at EL. -5.0 and a factor of safety of 1.5. For plates 121, 122, and 124 the actual design (completed before Dec 87) was controlled by the S-CASE, F.S. = 1.5, and canal level at EL -5.0. The design on plate 123 (completed after Dec 87) was based on the S-CASE F.S. = 1.2 with canal level at EL -5.0; S-CASE F.S. = 1.5 with canal level at EL 0.0; or the 3:1 penetration to head ratio. All the braced walls and tieback walls have been constructed. The tip penetrations shown on plates 122-124 are deeper than the minimum required tip penetration (Encl 3). Intermediate tip penetrations were used to reduce the anchor forces.

e. Comment e. Para. 39. Concur.

f. Comment f. Para. 41b. Reference Para. 3 of CEMRC-ED-GS dated 24 July 89. The 2-CASE F.S. = 1.25 applies only for waveloads on hurricane protection levees. No waveloads occur in the canal.

g. Comment g. Para. 68.

(1) Concur. The narrative description and explanation of the cost estimates is added as follows:

The unit prices and estimated costs were obtained with the use of a personal computer software system developed within NOD. This is the same computer software system utilized in preparing Government estimates for advertised construction projects.

The contingency percentage for each item was established by evaluating the uncertainty of both the quantity and costs for each item with the appropriate design engineer and cost engineer. Such evaluation was further assisted by the historical data on many cost items as this project incorporates a majority of items commonly found on hurricane protection projects within NOD.

Since the project is located in the metropolitan New Orleans area, accessibility presents no problem. Similarly, the materials for this project (including the steel and concrete products) are readily available. And as mentioned previously, the construction work is conventional and similar to such work throughout NOD which results in no unusual pricing.

(2) We incorporated the project contingencies in the cost estimate for account 30 and 31 and the work for Pumping Station No. 6. The net project contingency for the east and west floodwall work is approximately 13 percent. The total project contingency is 15 percent.

h. Comment h. Para. 69.

(1) Concur. The east-side work is currently under construction. The Orleans Levee Board's contractor is driving the piling to the depths specified in the DM. Approximately 6 to

7 feet of sheetpile stick-up above the levee crown is incorporated in the design to provide interim protection.

(2) See response c. above.

(3) The levee board elected to take the lead and commit these expenditures at this time to achieve savings because the New Orleans Sewerage and Water Board (NOSWB) planned to deepen and widen the canal to meet their drainage needs. The work proposed by NOSWB required modification to the existing levees and floodwalls. Under their permit, NOSWB is required to maintain the status quo for the existing levees including overall stability and level of protection. Since this involved pulling and driving new sheetpiling, it is obviously more cost effective to combine the two projects and replace the sheetpiling to the High Level Plan criteria. OLB elected to expend these funds even though they currently have credits in excess of those required. However, they see no reason to cap the sheetpiling at this time since it would only increase their share of the project cost. With the remaining work in the Orleans unit, it appears likely that OLB will have contributed in excess of their 30 percent share.

i. Comment i. Table 1. Concur. We will furnish the final design watersurface profile when the bridge designs conditions are known. In developing the designs for the bridges, we will insure that bridge losses are not excessive so as to cause the flowline to encroach into the floodwall freeboard.

j. Comment j. Plate 15. The stress levels in this portion of the concrete cap are below 3 ksi. We designed the lower section of the concrete cap with sufficient reinforcement to prevent cracking.

k. Comment k. Plates 80, 82, 84, 92 and 97. The minimum spacing between active and passive failure wedges is based on engineering judgment. We generally use a distance equal to about 0.7 and 1.0 of the height of the wedge, whichever is more prudent.

l. Comment l. Plates 94, 95, 117, 118 and 122. Concur. The W/L stationing on these plates applies to the area between W/L Station 570+00 and the pumping station. We will revise the stationing.

CELMN-ED-SP (1110-2-1150a)
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal

m. Comment m. Plates 121 through 124. Concur. Revised plates 121 through 124 are enclosed as Encl 3.

n. Comment n. Appendix D. The 27 percent contingency is listed as a project contingency. This contingency rate represents a weighted average of all the contingencies in the estimate. The 30 and 31 accounts have their respective contingencies built in to the summarized figures given in the table. We do not think that a more detailed presentation is necessary since the table is for an alternative plan which is not recommended. The rather high overall project rate is supportable owing to the more complex phased construction necessary to build the butterfly valve structure in the canal and at the same time not interrupt pumping operations.

FOR THE COMMANDER:

3 Encls
Added 2 encls
2-3. as

W. EUGENE PICKNER
Chief, Engineering Division

E-10064
STEPPS
CELMN-ED-SI
BARTON
CELMN-ED-SI
SHELTON
CELMN-ED-S
PICCIOLA
CELMN-ED-F
MARSALONE
CELMN-ED-D
SOILEAU
CELMN-ED-I
PICKNER
CELMN-ED

DEPARTMENT OF THE ARMY
MISSISSIPPI RIVER COMMISSION, CORPS OF ENGINEERS
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CEMRC-ED-GS

23 DEC '87

MEMORANDUM FOR: Commander, New Orleans District, ATTN: CELMV-ED-F

SUBJECT: Sheet Pile Wall Design Criteria

1. The final "E-99 Sheet Pile Wall Load Test Report (ref 5c, encl 3)" is enclosed along with our responses (ref 5b, encl 2) to the CELMN-ED comments in the 1st End to ref 5a (encl 1).

2. Also enclosed is a CEWES-GE memorandum which summarizes the results of a finite element study (see 2d and 3d endorsements to ref 5a). In short, this study indicates that, for a sheet pile wall driven into a levee founded on very soft to soft clays, the majority of lateral sheet pile movement during flood loading will likely be due to deep seated foundation movement and not due to sheet pile flexural deflection. Driving the sheet pile deeper has little effect on overall levee stability, or after some limiting depth, on flexural deflection at the top of the wall. This data reinforces the conclusions reached in the E-99 report that excessive sheet pile penetrations are not warranted.

3. Based on the E-99 report and the CEWES-GE study, the following design criteria have been developed and should be followed to determine the required penetration for sheet pile floodwalls founded in soft clays.

Q Case

F.S. = 1.5 with water to flowline or SWL
F.S. = 1.25 with water to freeboard (net levee grade) for river levees or with SWL and waveload for hurricane protection levees
F.S. = 1.0 with SWL + 2' Freeboard - (Never use more than 2' freeboard

S-Case

F.S. = 1.2 with water to flowline or SWL + waveload (if applicable) for hurricane protection levees
F.S. = 1.0 with water to freeboard (net levee grade) for river levees

Select the maximum penetration from the above analyses. Since the E-99 test section, at maximum head, had about a 3:1 penetration to head ratio*, the above design procedure should primarily be applied to this or greater penetration to head ratios. The primary intent of this revised criteria is to prevent excessive sheet pile penetrations which do not improve either sheet pile or overall levee stability. If the penetration to head ratio is less than about 3:1, increase it to 3:1 or to that required by the S-case, F.S. = 1.5, whichever results in the least penetration. Further study may allow this restriction to be eliminated.

* use SWL or flowline to calculate head for penetration to head ratio.

23 DEC 87

4. In the past, we understand that LMNED has used an arbitrary limiting deflection criteria of 3 inches of estimated lateral flexural deflection between the tip of the sheet pile and the top of an I-wall to select the appropriate sheet pile section and to determine the suitability of an I-wall in general. However, as discussed in para 8 of encl 4, the majority of the actual lateral movement of a sheet pile driven into a levee founded on a soft foundation will most likely be due to deep seated movements in the levee foundation and not flexural deflection. In addition, flexural deflection cannot be computed with any degree of accuracy using conventional analyses (encl 3). While it is recognized that there may be many cases for which the current criteria is appropriate, you should reevaluate the use of the 3-inch maximum lateral deflection criteria. On the E-99 test wall, the inclinometers attached to the sheet piles indicated as-driven inclinations as great as 8 inches from the vertical between the tip and top of the sheet pile. Since final inclinations as great as 11 inches were recorded upon loading and were not noticable, the 3-inch deflection criteria based on esthetics appears unreasonable. You should consider on a case by case basis what would be an acceptable maximum deflection in a given project. For example, in a reach of wall where a transition section exists between a relatively flexible I-wall and a somewhat stiff T-wall, it may be desirable to minimize I-wall deflections. However, since encl 4 indicates that increasing the section and penetration of the sheet pile may have little effect on overall wall movement, it may be more prudent to provide a waterstop as flexible as possible between I-walls and pile founded structures. Conversely, if the project contains a long reach of I-wall, then deflections in excess of 3 inches may be acceptable, assuming of course that allowable flexural stresses have not been exceeded and that the wall is otherwise stable. Of primary importance for limiting I-wall movement for I-walls founded in levees, in addition to adequate embedment, is to ensure adequate overall levee stability.

5. Reference:

a. CEMRC-ED-GS letter dated 11 Sep 86, entitled "E-99 Sheet Pile Wall Load Test, Draft Report" and endorsements thereto (encl 1, correspondence only).

b. Responses to CELMN-ED comments in 1st End to the letter referenced above (encl 2).

c. E-99 Sheet Pile Wall Load Test, Final Report (encl 3). Bound copies of this final report will be furnished at a later date.

d. CEWES-GE memorandum dated 10 Dec 87, entitled "Development of Finite Element-Based Design Procedure for Sheet Pile Walls" (encl 4).

FOR THE PRESIDENT OF THE COMMISSION:

FRED H. BAYLEY III
Chief, Engineering Division

4 Encls
as

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CEMRC-ED-GS (1105-2-10c)

24 JUL '89

MEMORANDUM FOR Commander, New Orleans District, ATTN: CELMV-ED-F

SUBJECT: Sheet Pile Wall Design Criteria

1. Reference:

a. CEMRC-ED-GS letter, subject as above, dated 23 Dec 87.

b. CELMN-ED-DD letter, entitled "Phasing in of New I-Wall Design Criteria into NOD's Design/Construction Program," dated 26 Jan 88, and endorsements thereto.

c. CELMN-ED-DD letter, entitled "I-Wall Deflection," dated 18 Nov 88 and endorsements thereto.

d. WES final report entitled "Development of Finite Element-Based Design Procedure for Sheet Pile Walls" (encl 1).

2. The first two referenced letters (ref 1a and 1b) set forth revised criteria for determining the penetration of sheet pile floodwalls founded in soft clays. The third letter (ref 1c) primarily involved discussions concerning estimating sheet pile deflections and design of I-walls to withstand these deflections. The purpose of this letter is to summarize the guidance for determining sheet pile wall penetrations, deflections, and moments based on the referenced letters and an evaluation of the referenced report.

3. The following criteria should be followed to determine the penetration of sheet pile floodwalls founded in soft clays:

Q-Case

F.S. = 1.5 with water to flowline or SWL.

F.S. = 1.25 with water to flowline plus approved freeboard for river levees or with SWL and waveload for hurricane protection levees.

F.S. = 1.0 with SWL plus 2-ft freeboard for hurricane protection levees.

S-Case

F.S. = 1.2 with water to flowline or SWL and waveload. If a hurricane protection floodwall has no significant waveload, determine the penetration using Q-case criteria only.

F.S. = 1.0 with water to flowline plus approved freeboard for river levees.

Select the maximum penetration from the applicable cases above using a limit equilibrium analysis such as CANWAL. In some cases, especially Q-case penetrations derived for low heads, the theoretical required penetrations could be minimal. In order to ensure adequate penetration to account for unknown variations in ground surface elevations and soil conditions, penetrations should be arbitrarily increased, as necessary, to achieve a penetration to head ratio (for flowline or SWL) of about 2.5 to 3:1. Engineering judgement should be exercised in selecting appropriate loading cases and penetration to head ratios. For certain projects, penetration to head ratios of less than 2.5 to 3:1 may be appropriate.

4. Moment and shear forces computed for design of the sheet pile wall sections should be based on the most critical loading case set forth in para 3 above and a conventional limit equilibrium analysis such as CANWAL. Based upon comparisons presented in reference 1d using the Finite Element Method and soft clay conditions, there is no significant increase in moment due to increasing pile penetration (see plots in reference 1d, pages A-26, A-28, and A-30). Consequently moment and shear forces computed for design of sheet pile wall sections need only be based upon the critical load case set forth in paragraph 3 above and the resulting pile penetration, even if the selected penetration is greater than the computed required penetration. As stated in reference 1d, displacements of sheet pile walls founded in soft clays are likely more the result of deep seated soil movements than due to flexural deflection of the sheet pile. Therefore, in addition to the calculation of flexural deflection based on the critical loading case and the limit equilibrium analysis, the finite element derived recommendations outlined on page 47 of ref 1d can be used to help estimate total wall deflections. Of course, the closer the actual project site conditions are to those assumed in the WES report, the more applicable the finite element derived deflections. In any case, estimated sheet pile deflection should not control the selection of the sheet pile section for walls founded in soft clays. A flexible connection should be designed to accommodate the estimated relative deflections between I-walls and adjacent pile or soil founded monoliths.

5. It should be noted that the finite-element estimated wall deflections in the WES final report are somewhat less than those in its draft report. This is primarily due to the selection of a higher "K" value (soil stiffness) to calibrate the finite element model to the E-99 field test data. Due to sensitivity of the computed and actual deflections to soil stiffness, the actual deflections experienced in the field can only be estimated with limited accuracy. If the I-wall/levee is designed for a minimum F.S. = 1.30 for sliding stability, deep seated foundation movements should not normally be excessive.

6. In future design reports and design memorandums, the following information should be shown on each I-wall Stability Plate:

a. Summary of load cases considered in the design.
b. Moment Diagram for controlling load case.
c. Shear Diagram for controlling load case.
d. Deflections computed by both CANWAL and the WES report method (if applicable.
e. Sample computations illustrating the selection of the required sheet pile section.

CEMRC-ED-GS
SUBJECT: Sheet Pile Wall Design Criteria

24 JUL '89

If space permits show items b and c on levee cross-section immediately adjacent to sheet pile soil pressure diagram.

FOR THE PRESIDENT OF THE COMMISSION:

Encl

FRED H. BAYLEY III
Chief, Engineering Division

# PART V: RECOMMENDATIONS

63. Based on the findings outlined in Part IV, it is recommended that sheet-pile wall design be based on the static equilibrium of the sheet pile-levee system. The stability of the levee would be based on a conventional analysis preferably using a circular arc method (although both circular arc and wedge-shaped cases should be checked). This analysis would determine a maximum water loading that could be tolerated. The pile embedment would be determined using the conventional criteria for static equilibrium of a cantilever wall (i.e. by CANWAL). This analysis would determine the embedment needed. The strength parameter to be used for the analysis should be consistent with the unconsolidated undrained (end-of-construction) condition (i.e. $c = S_u$ and $\phi = 0$). If wall displacement is an important design parameter, the semi-empirical technique based on Figure 21 can be used. If site conditions differ significantly from those considered in this report, displacements should be determined by a complete finite element analysis unless the safety factor for deep-seated movement is high. If the safety factor for the foundation (as computed by slope stability methods) is high, displacements can be computed by CANWAL based on the embedment corresponding to a safety factor of 1.0. It is estimated from Figure 19 that the safety factor for foundation stability must be well above 2.0 before the displacements computed by CANWAL are appropriate. Because of complicating factors there is no known general procedure that can be used to correct the maximum moments computed by CANWAL at this time.

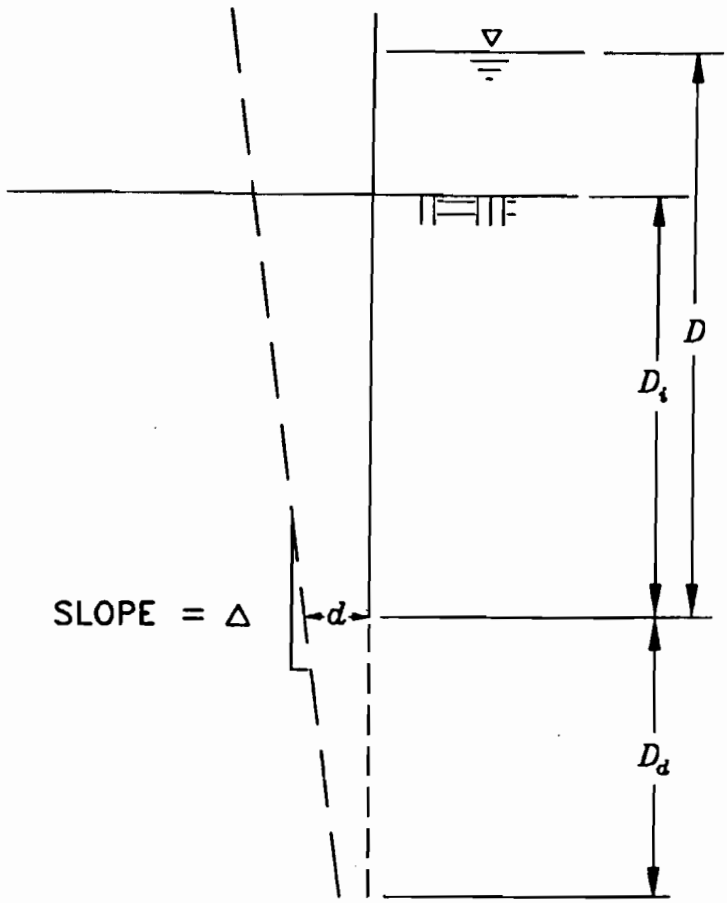


Figure 21. Schematic of slope in pile and movements at pile tip due to movements in the foundation

61. The procedure amounts to adding a "correction" to the displacement computed by CANWAL. First, the embedment and displacement corresponding to a safety factor of 1.0 are computed by CANWAL. The computed embedment depth is $D_i$ and the total length of pile is $D$ as shown in Figure 21. As discussed above, this displacement corresponds to the correct moment distribution. Second, the additional embedment depth $D_d$ needed to obtain the required safety factor is computed using CANWAL. Next the appropriate plots in Figures 22 to 25 are used to determine the displacement and slope at the pile tip. The displacement at the top of the pile is thus the sum of the CANWAL displacement and the quantity $d + (\Delta \times D)$.

## Conclusions

62. Task III was to perform detailed analyses and develop recommendations for new sheet-pile wall design procedures. The analyses were performed using the E-105 sheet pile-levee



From Ref 1.d.

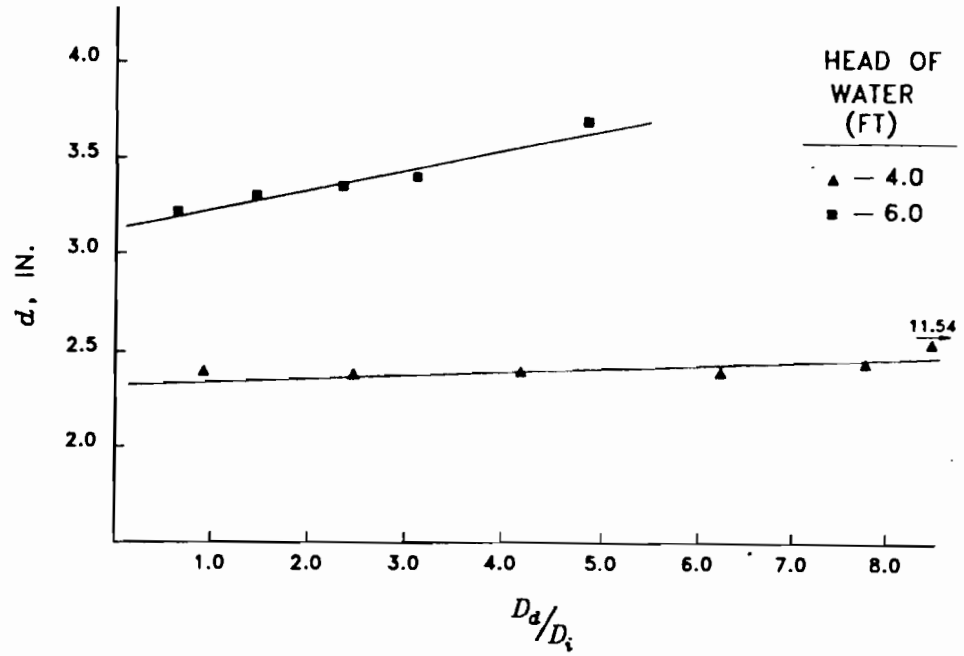


Figure 22. Movement at pile tip due to movements in foundation for the E-105 "weak" soil profile

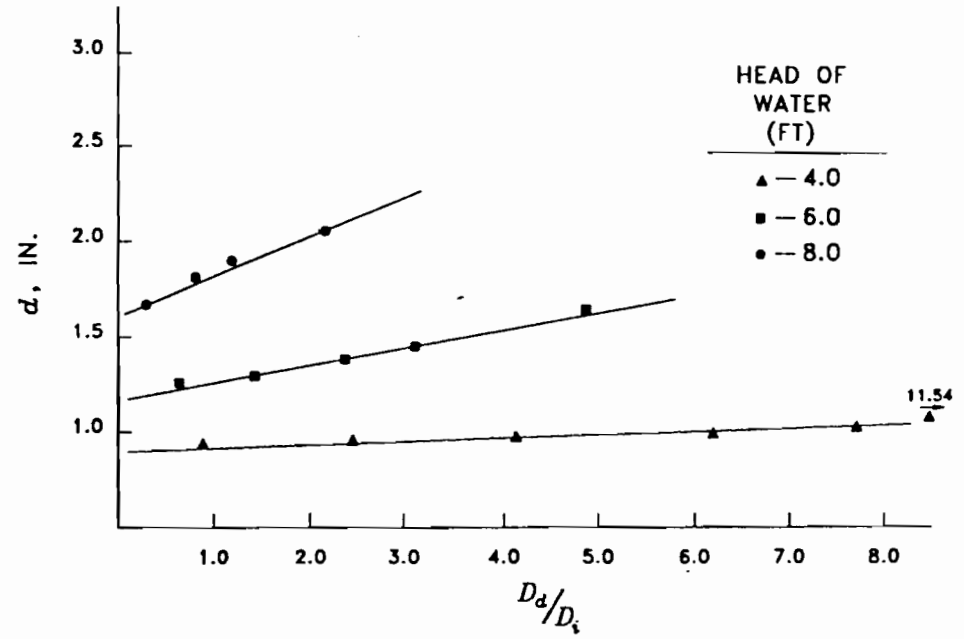


Figure 23. Movement at pile tip due to movements in the foundation for the E-105 "strong" soil profile

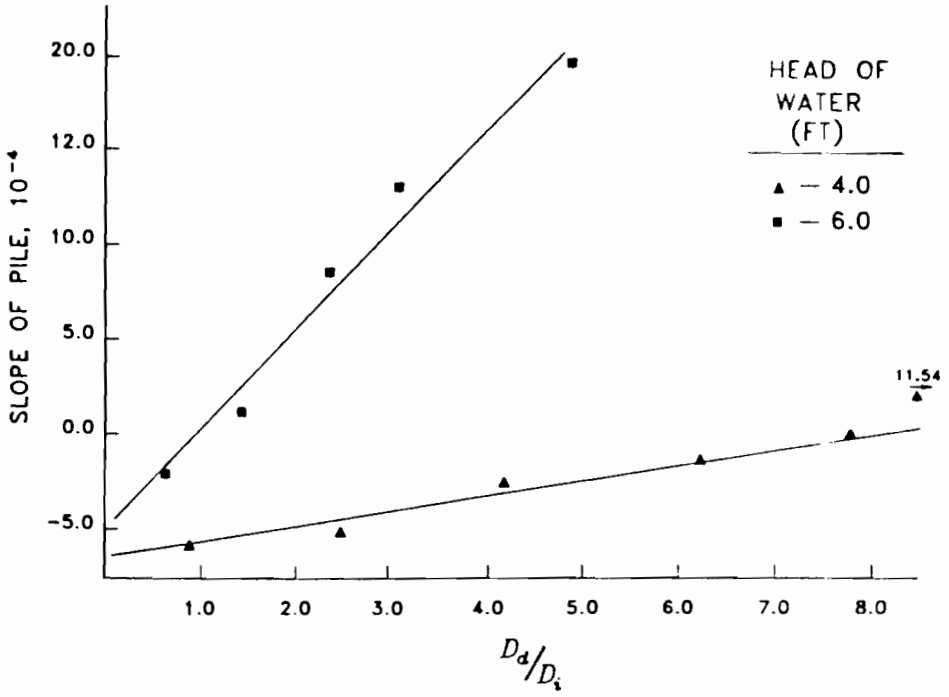


Figure 24. Slope in pile at pile tip due to movements in the foundation for the E-105 "weak" soil profile

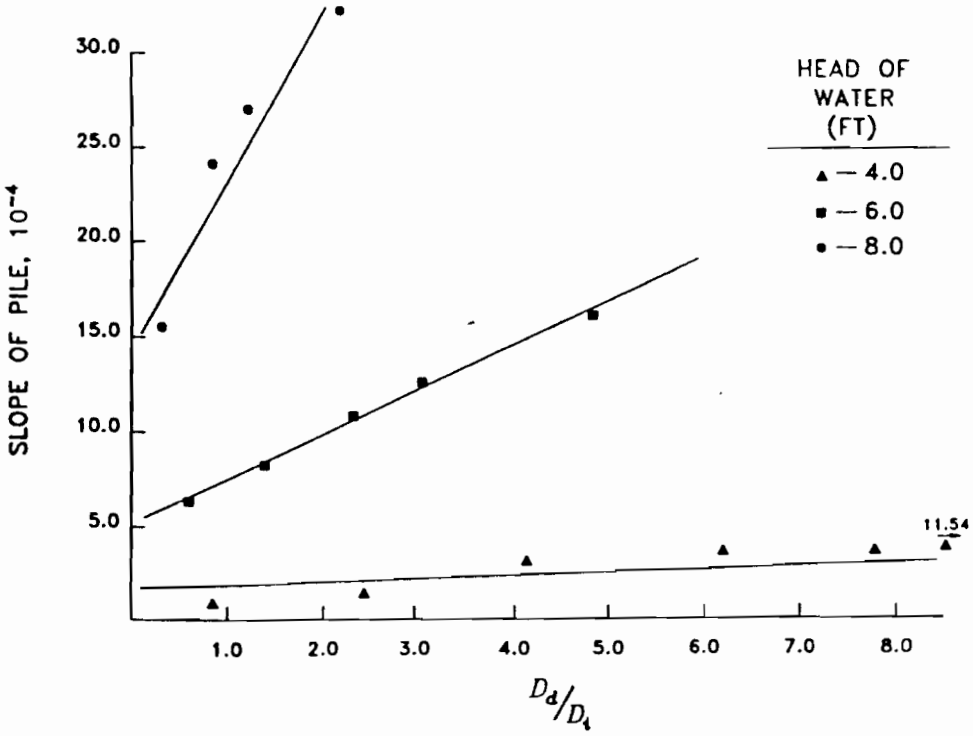


Figure 25. Slope in pile at pile tip due to movements in the foundation for the E-105 "strong" soil profile

profile, Figure 9. An analysis of the E-105 profile has been completed and the following basic conclusions have been reached.

a. Deep-seated movements in the levee foundation control the magnitude of sheet-pile deflection, particularly in soft soils. As a result, the height of water loading that can be sustained by a particular I wall is controlled by the stability of the foundation, as determined by a slope stability analysis.

b. The stability of the levee implied by the displacements is consistent with the safety factor computed by limit-equilibrium methods.

c. Increased sheet-pile penetration does not improve the stability of the levee.

d. The stability of the sheet pile relative to overturning, as implied by computed displacements, is consistent with the safety factors computed by CANWAL.

e. Penetration of the sheet pile below that needed to meet requirements for resistance against overturning does not improve performance of the sheet pile.

f. Pile stiffness has little effect on total displacements.

g. Deflection of the sheet-pile wall, as conventionally determined using the CANWAL program, is a poor criterion for design of sheet-pile walls because movements are caused by shear deformation in the foundation and not the cantilever action of the pile.

h. The moments computed by CANWAL for a safety factor of 1.0 agree best with those obtained from the finite element analysis.

DEPARTMENT OF THE ARMY
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REPLY TO
ATTENTION OF:
CELMN-ED-SP (1110-2-1150a) 15 May 90

MEMORANDUM FOR Commander, Lower Mississippi Valley Division,
ATTN: CELMV-ED-PG

SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal

1. The subject design memorandum is submitted for review and approval, and has been prepared generally in accordance with the provisions of ER 1110-2-1150, dated November 1984.

2. A summary of the current status of the Clean Water Act, endangered species, Environmental Impact Statement (EIS), and cultural resources investigations is as follows:

a. Since the tentatively selected plan will not require the deposition of dredged and fill materials into the waters of the U.S., a Section 404(b)(1) Evaluation is not necessary.

b. Based on studies and investigations at this stage of designs, the proposed action is not likely to jeopardize the continued existence of any endangered species or result in the destruction or adverse modification of the critical habitats of such species.

c. A final EIS for the barrier plan for the subject project was filed with Council on Environmental Quality (CEQ) on 17 January 1975. A final supplement to this EIS was filed with the Environmental Protection Agency (EPA) on 7 December 1984. An environmental Assessment addressing the recommended plan (parallel protection) was mailed to the public on 27 March 1990.

d. The project area consists of an existing levee corridor on Post-1930 reclaimed land, and the artificial channel of the 17th Street Outfall canal. No cultural resources are recorded in the vicinity of the proposed work and no cultural resource surveys are warranted.

3. In accordance with LMNED-TS memorandum dated 5 February 1981, this report has been reviewed by the District Security Officer. There were no comments to be incorporated in the report.

CELMN-ED-SP (1110-2-1150a)
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal

4. Reference CECW-EP, multiple memorandum dated 16 February 1990: GDM Review Process. No adverse issues were raised in the Administration's clearing process or no known policy changes have occurred which would concern OMB relative to the planning and designs presented in this DM. The status of the LCA's for the project feature addressed in this DM is discussed in paragraph 8, Volume I. Cost sharing for the project remains as originally authorized i.e., 70% Federal and 30% non-Federal. Preparation of this DM is in compliance with the most recent guidelines and policies covering plan formulation and engineering design. House Document No. 231, 89th Congress served as the basis for the feasibility report on the Lake Pontchartrain project and the subsequent project authorization, Public Law 298, 89th Congress, 1st Session. Higher authority approval of the July 1984 reevaluation report entitled, "Lake Pontchartrain, Louisiana and Vicinity Hurricane Protection Project", is the vehicle which authorized the current "High Level Plan". That document now serves as the feasibility report for the Lake Pontchartrain project.

5. This report was scheduled to be submitted to LMVD by 31 March 1990. This delay will not cause a delay in the start of construction.

6. Approval of this Design Memorandum as a basis for preparation of plans and specifications is recommended.

FOR THE COMMANDER:

W. EUGENE TICKNER
Chief, Engineering Division

Encl
(30 cys fwd sep)

CELMV-ED-PG (CELMN-ED-SP/15 May 90) (1105-2-10c) 1st End Cardwell/ts/5925
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal

CDR, Lower Mississippi Valley Division, Vicksburg, MS 39181-0080

08 AUG '90
FOR Commander, New Orleans District, ATTN: CELMN-ED-SP

The subject DM is approved subject to the satisfactory resolution of the following comments.

a. Para 7k. The interest rate shown as 3.225 percent per annum should be 3.125 percent per annum.

b. Para 14 and Appendix DD, page DD-9. It is not clear from this paragraph and the design computations in Appendix DD whether a horizontal load condition will exist at the Veterans Highway Bridge similar to that described in the Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19 - General Design, Orleans Outfall Canal, 11 Aug 1988, for the Robert E. Lee Bridge modification. If such a loading condition does in fact exist, it should be addressed and the design computations should be furnished for review. In addition, the guidance contained in paragraph c of the 3rd endorsement to DM No. 19, dated 10 Mar 89, should be evaluated for possible applicability in determining pile fixity for the Veterans Highway Bridge if a similar horizontal loading exists.

c. Para 29b. Due to the critical nature of this project and the close proximity of the adjacent canal, a minimum penetration to head ratio of 3 to 1 should be used for sheet pile design for this project. We note that the 3 to 1 minimum ratio has been used on less critical projects in the New Orleans area. In addition, the sheet piles shown on I-wall penetration analysis Plates 101 to 105 and 110 to 113 will serve as permanent bulkheads retaining as much as 4 ft of soil. To ensure adequate bulkhead stability toward the floodside, these sheet pile bulkheads should be analyzed using the "S" case soil strengths, a factor of safety of 1.5, and a canal level of el 0.0. Based on the above, the sheet pile penetration in this DM should be increased as necessary. If there is a potential for erosion at the floodside toe of the bulkheads, stone protection should be considered.

d. Para 29c. Also refer to the tied back and braced wall analyses on Plates 121-124. These analyses are based on the "C" case soil strengths, a canal level at el -5.0, and a factor of safety of 1.5. Since these sheet pile bulkheads permanently retain soil, they should also be analyzed using "S" soil strengths, a factor of safety of 1.5, and at the normal canal level of el 0.0.

e. Para 39. CECW-ED Circular No. 1110-2-267, subject: Strength Design for Reinforced Concrete Hydraulic Structures, 31 Jan 90, has superseded ETL 1110-2-312 referenced in this paragraph. For work not previously completed, the guidance in the circular should be followed.

f. Para 41b. Provide the rationale for not considering the C-case with F.S. = 1.25, as shown in paragraph 29b. If this case is applicable, wave loads should be incorporated into the loadings for the floodgates and gate monoliths.

g. Para 68.

(1) The text does not contain a narrative description and explanation of the cost estimate. This description should immediately precede the cost estimate and should contain the cost engineers methods of cost estimate preparation; i.e., means of establishing unit prices, quantities, etc.; the rationale for establishing contingencies; areas of uncertainties in the estimate, etc.

(2) The chosen plan has a net project contingency of less than 10 percent on the east and west side levees and floodwalls and no contingencies identified for Accounts 30 and 31 and the work for Pumping Station No 6. A project contingency level of 10 percent is normally acceptable only when the estimate was based on completed plans and specifications. You should identify contingencies on Accounts 30 and 31 and the work at Pumping Station No. 6 and review the contingencies set on the remaining project. A total contingency of less than 15 percent for this project should not be used unless plans and specifications are completed. The setting of contingencies should be fully explained in the cost estimate write-up discussed in comment g(1) above.

h. Para 69.

(1) This schedule indicates that the construction contract for the east side floodwall will be awarded in Oct 95. However, we understand that the Orleans Levee Board has already awarded a contract to drive sheet pile for the east bank floodwall and also to perform some dredging work, and the work under this contract was to commence in early Jul 90. These sheet piles are to be driven full length and not capped with concrete until the scheduled contract award in Oct 95. We understand the plans and specifications for the current sheet pile contract were reviewed and approved by you and the sheet pile lengths specified are the same as shown in this DM.

(2) Compliance with comment c above will result in additional sheet pile penetrations in some reaches over that shown in this DM. The fact that a construction contract was awarded for the east side sheet pile work prior to our review of this DM results in an undesirable situation for this office and the Corps. The current Orleans Levee Board contract should either be modified to provide the additional lengths or the sheet piles should be driven as shown in the DM and later driven to the revised penetration just prior to capping. The Orleans Levee Board should be advised that there is some risk involved with waiting 5 years to achieve the revised sheet pile penetrations. The sheet pile lengths for the west side floodwall should be revised prior to preparation of plans and specifications.

(3) It is not clear why the sheet piles are being driven full length and not capped under the current Orleans Levee Board floodwall contract. Since the existing levee is either being degraded or left intact, no settlement should occur after driving the sheet pile. Therefore, the considerable additional expense incurred by driving the sheet piles full length at this time would not appear justified. If you conclude that it is not possible to cap the floodwall under the current Orleans Levee Board floodwall contract, action should be taken to cap the floodwall as soon as possible in order to reduce the period in which the floodwall will be exposed to the risk mentioned in the paragraph above.

i. Table 1. The water surface elevations and head losses are shown for the various bridge conditions under existing and future pump discharge capacities; however, the design flowline and freeboard are not indicated. These should be presented when the final bridge conditions are known.

j. Plate 15. If not previously accomplished suggest you check to ensure by analysis that cracking in the lower portion of the concrete I-wall section shown in the center of the plate will not occur. Our concern with cracking stems from the restraint that will be encountered in the lower section, the abrupt change in width of the concrete between the lower section and upper section, and our unfamiliarity and inexperience with this particular wall design configuration. In addition consider seeking the advice of the concrete experts at WES for their opinions pertaining to this potential cracking and possible remedies that could economically be incorporated in the design to alleviate this concern.

k. Plates 60, 82, 84, 92, and 97. Some of the active and passive wedges shown on these plates are too close together to represent any possible failure planes which might develop. In future reports, the active and passive wedges should be separated at least by a distance equal to the height of the active wedge.

l. Plates 94, 95, 117, 118 and 122. The soil strengths and stratification used on these plates appear to be based on that shown on Plate 56 for the reach "Pumping Station to Sta 670+00." However, the W/L stationing on these plates seem to represent areas lakeside of Hammond Highway. We understand that the W/L stationing on these plates is in error and will be revised as necessary.

m. Plates 121 through 124. The walls shown on these plates should be evaluated for the anchor forces and resulting stresses on the anchors and the sheet piling should be checked using the conditions shown except change the water level on the canal side to el 0.0 NGVD and use soil properties for the 'S' case and a F.S. = 1.5.

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n. Appendix D. The construction cost contingency is shown as 27 percent with no contingency shown on Accounts 30 and 31. This cost estimate should have some explanation of the assumptions made, use of contingencies, etc.

FOR THE COMMANDER:

Encl wd

FRED H. BAYLEY III
Chief, Engineering Division

CF (w/10 cys encl 1):
CECW-EP