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COMMUNITY DEVELOPMENT

TUKWILA SMP
LA PIANTA RIVER PARCELS

TAX PARCEL #		SITE AREA (KING CO.)		2009 KING CO. PROPERTY VALUATION		
		Square Feet	Acres	Land	Improvements	Total
352304	9116	232,770	5.34	$698,300	$227,500	$925,800
352304	9055	70,594	1.62	$1,058,900	$2,560,600	$3,619,500
352304	9121	363,873	8.35	$2,183,200	$0	$2,183,200
352304	9018	434,155	9.97	$3,256,100	$4,146,100	$7,402,200
352304	9115	629,172	14.44	$4,718,700	$12,594,200	$17,312,900
352304	9036	209,151	4.8	$1,568,600	$19,500	$1,588,100
352304	9017	87,120	2.00	$653,400	$0	$653,400
352304	9041	1,126,897	25.87	$901,500	$0	$901,500
						$0
022204	9033	649,915	14.92	$519,900	$0	$519,900
022204	9040	37,296	0.86	$74,500	$0	$74,500
022204	9043	905,612	20.79	$1,811,200	$0	$1,811,200
022204	9057	382,456	8.78	$133,800	$0	$133,800
022204	9011	1,176,991	27.02	$882,700	$0	$882,700
022204	9015	2,112,224	48.49	$1,584,100	$0	$1,584,100
TOTALS		8,418,226	193.25	$20,044,900	$19,547,900	$39,592,800

EXHIBIT 12 DATE 4/20
PROJECT NAME Council Review SMP
FILE NO Update L06-088

King County Property Description for parcel number 3523049116 - Mozilla Firefox

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Assessor Information for parcel number 3523049116

Taxpayer name	LA PIANTA LLC
Mailing address	PO BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	18500 SOUTHCENTER PKWY 98188

Parcel number	3523049116
Tax Account number	352304911600
Levy code	2360
Jurisdiction	TUKWILA
Present use	Office Building
Appraised value	$925,800

Legal description

PAR D OF TUKWILA BLA #L02-029 REC #20021007900001 SD PAR LOCATED E 1/2 & SW 1/4 OF SD SEC

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number
07-01-2008	$0	KING COUNTY-FLOOD CONTROL ZONE DISTRICT	KING COUNTY	2353367
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410758

Parcel description

Property name	ENTERPRISE LEASING CENTER	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Office Building	Plat lot		Access	PUBLIC
Lot area	232,770 sq. ft. (5.34 acres)	Q-S-T-R	SW-35-23-4	Street surface	PAVED

Commercial building description

Building	1 of 1	Building description	OFFICE
Year built	1972	Predominant use	OFFICE BUILDING (344)
Stories	1	Gross sq. ft.	3,465
Building quality	AVERAGE	Net sq. ft.	3,465
Construction class	WOOD FRAME	Heating system	FORCED AIR UNIT
Building shape	Rect or Slight Irreg	Sprinklers	N
		Elevators	

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$698,300 (land) + $227,500 (improvements) $925,800 (total)	$698,300 (land) + $227,500 (improvements) $925,800 (total)
2008	TAXABLE	NONE OR UNKNOWN	$630,000 (land) + $220,200 (improvements)	$630,000 (land) + $220,200 (improvements)

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Assessor Information for parcel number 3523049055

Taxpayer name	LA PIANTA LLC
Mailing address	PO BOX 88028 TUKWILA WA 98138

Parcel number	3523049055
Tax Account number	352304905503
Levy code	2300
Jurisdiction	TUKWILA
Present use	Office Building
Appraised value	$3,619,500

Address(es) at this parcel: 18000 ANDOVER PARK W 98188

Legal description

LOT 1 OF CITY OF TUKWILA SHORT PLAT NO 86-45 SS RECORDING NO 8609081152 AS AMENDED BY CITY OF TUI
PORTION OF SW 1/4 OF NE 1/4 AND OF GOV LOT 2 - BEGIN NE CORNER OF SW 1/4 OF NE 1/4 TH S 07-44-50 W 78.
CENTER BEARING S 82-15-04 E RADIUS OF 50 FT THRU C/A OF 84-24-55 ARC DISTANCE OF 73.67 FT TH S 87-50-0
06-06-46 ARC DISTANCE OF 35.83 FT TAP OF CURVE CENTER BEARING N 73-34-06 W TH ALONG CURVE TO RIGHT
COMPOUND CURVE TH ALONG CURVE TO RIGHT RADIUS OF 409.256 FT THRU C/A OF 58-14-17 ARC DISTANCE OF
1 OF CITY OF TUKWILA LOT LINE ADJ NO 93-0085 RECORDING NO 9311301961 ... LESS RR R/W ESMT IN NE 1/4 A

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number	Recording numbe
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410756	199412300689

Parcel description

Property name	SEAGLE BLDG #862	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Office Building	Plat lot		Access	PUBLIC
Lot area	70,594 sq. ft. (1.62 acres)	Q-S-T-R	NE-35-23-4	Street surface	PAVED

Commercial building description

Building	1 of 1	Building description	OFFICE
Year built	1987	Predominant use	OFFICE BUILDING (344)
Stories	2	Gross sq. ft.	26,670
Building quality	AVERAGE	Net sq. ft.	25,336
Construction class	MASONRY	Heating system	WARMED AND COOLED AIR
Building shape	Rect or Slight Irreg	Sprinklers	Y
		Elevators	

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$1,058,900 (land) + $2,560,600 (improvements) $3,619,500 (total)	$1,058,900 (land) + $2,560,600 (improvements) $3,619,500 (total)
2008	TAXABLE	NONE OR UNKNOWN	$988,300 (land) + $2,517,100 (improvements)	$988,300 (land) + $2,517,100 (improvements)

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King County Property Description for parcel number 3523049121 - Mozilla Firefox

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Assessor Information for parcel number 3523049121

Taxpayer name	LA PIANTA LLC
Mailing address	PO BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	None

Parcel number	3523049121
Tax Account number	352304912103
Levy code	2360
Jurisdiction	TUKWILA
Present use	Vacant(Industrial)
Appraised value	$2,183,200

Legal description

LOTS 3 & 4 OF CITY OF TUKWILA SHORT PLAT NO 86-45 SS RECORDING NO 8609081152 AS AMENDED BY CITY OI
DAF - PORTION OF SW 1/4 OF NE 1/4 AND OF GOV LOT 2 & 5 - BEGIN NE CORNER OF SW 1/4 OF NE 1/4 TH S 07-44-5
ALONG CURVE TO LEFT RADIUS OF 409.26 FT THRU C/A OF 58-14-17 ARC DISTANCE OF 415.99 FT TAP OF COMPOI
00-25-39 ARC DISTANCE OF 4.28 FT TAP ON S MGN OF S 180TH ST TH NELY ALONG SAID S MGN ON CURVE TO LEI
ARC DISTANCE OF 96.13 FT TH S 36-31-53 W 135.86 FT TH ALONG CURVE TO LEFT RADIUS OF 430 FT THRU C/A OF
CURVE TO LEFT RADIUS OF 800 FT THRU C/A OF 18-44-11 ARC DISTANCE OF 261.61 FT TH S 03-19-15 W 141.75 FT
DISTANCE OF 225.34 FT TH S 07-00-35 W 20.08 FT TH S 09-03-25 W 579.24 FT TH S 06-44-45 W 82.23 FT TH N 66-5
BEARING N 71-16-17 W RADIUS OF 2530 FT THRU C/A OF 10-58-47 ARC DISTANCE OF 484.83 FT TH N 07-44-56 E 6'
RECORDING NO 9311301961 --- LESS RR R/W ESMT IN NE 1/4 & SE 1/4 (OPERATING PROPERTY)

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax num
07-01-2008	$0	KING COUNTY-FLOOD CONTROL ZONE DISTRICT	KING COUNTY	2353367
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410756

Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	363,873 sq. ft. (8.35 acres)	Q-S-T-R	NE-35-23-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$2,183,200 (land) + $0 (improvements) $2,183,200 (total)	$2,183,200 (land) + $0 (improvements) $2,183,200 (total)
2008	TAXABLE	NONE OR UNKNOWN	$2,160,800 (land) + $0 (improvements) $2,160,800 (total)	$2,160,800 (land) + $0 (improvements) $2,160,800 (total)
2007	TAXABLE	NONE OR UNKNOWN	$2,160,800 (land) + $0 (improvements)	$2,160,800 (land) + $0 (improvements)

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Assessor Information for parcel number 3523049018

Taxpayer name	LA PIANTA LLC	Parcel number	3523049018
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	352304901809
		Levy code	2360
		Jurisdiction	TUKWILA
		Present use	Warehouse
		Appraised value	$7,402,200
Address(es) at this parcel	18323 ANDOVER PARK W 98188		

Legal description

PORTION OF GOV LOT 5 IN SE 1/4 OF SECTION 35-23-04 - BEGIN NW CORNER OF GOV LOT 5 TH S 87-56-03 E 1141
DIRECTION ALONG CURVE TO RIGHT CENTER BEARING N 73-28-16 W RADIUS OF 2470 FT THRU C/A OF 09-57-44 A
N 69-15-04 W 180 FT TH S 20-44-56 W 511.97 FT TH S 66-31-28 E 253.17 FT TH S 29-14-18 W 75.38 FT TH S 66-31-
DEFINED UNDER K C RESOLUTION NO 31833 UNDER AUDITOR FILES 6014672 & 6027015 TH NLY ALONG SAID EAS
22-32-00 E 115.31 FT TH ON CURVE TO RIGHT RADIUS 560 FT THRU C/A OF 23-39-57 ARC DISTANCE 231.31 FT TH
25-46-59 ARC DISTANCE 207 FT TH ALONG CURVE TO LEFT RADIUS 1000 FT THRU C/A OF 06-47-12 ARC LENGTH 1
POINT OF CURVATURE TH IN NLY DIRECTION ALONG CURVE TO LEFT RADIUS 2530 FT THRU C/A OF 02-33-00 ARC
TO POINT OF BEGINNING - PARCEL B OF CITY OF TUKWILA BOUNDARY LINE ADJUSTMENT NO L99-0008 RECORDI

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Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax num
07-01-2008	$0	KING COUNTY-FLOOD CONTROL ZONE DISTRICT	KING COUNTY	2353387
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410756

Parcel description

Property name	SEGALE BUSINESS PARK	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Warehouse	Plat lot		Access	PRIVATE
Lot area	434,155 sq. ft. (9.97 acres)	Q-S-T-R	SE-35-23-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$3,256,100 (land) + $4,146,100 (improvements) $7,402,200 (total)	$3,256,100 (land) + $4,146,100 (improvements) $7,402,200 (total)
2008	TAXABLE	NONE OR UNKNOWN	$2,822,000 (land) + $4,358,200 (improvements) $7,180,200 (total)	$2,822,000 (land) + $4,358,200 (improvements) $7,180,200 (total)
2007	TAXABLE	NONE OR UNKNOWN	$2,822,000 (land) + $2,799,800 (improvements) $5,621,800 (total)	$2,822,000 (land) + $2,799,800 (improvements) $5,621,800 (total)

Related resources

King County Assessor: Submit a request to correct information in this report

King County Assessor: eReal Property Report (PDF format requires Acrobat)

King County Assessor: Quarter Section Map (PDF format requires Acrobat)

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Assessor Information for parcel number 3523049115

Taxpayer name	LA PIANTA LLC	Parcel number	3523049115
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	352304911501
		Levy code	2360
		Jurisdiction	TUKWILA
		Present use	Warehouse
		Appraised value	$17,312,900
Address(es) at this parcel	5811 SEGALE PARK - C DR 98188		

Legal description

PAR E OF TUKWILA BLA #L02-029 REC #20021007900001 SD PAR LOCATED E 1/2 & SW 1/4 OF SD SEC

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax numbe
07-01-2008	$0	KING COUNTY -FLOOD CONTROL ZONE DISTRICT	KING COUNTY	2353367
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410756

Parcel description

Property name	DISTRIBUTION WAREHOUSE	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Warehouse	Plat lot		Access	PRIVATE
Lot area	629,172 sq. ft. (14.44 acres)	Q-S-T-R	SW-35-23-4	Street surface	PAVED

Commercial building description

Building	1 of 1	Building description	BLDG # 981
Year built	2000	Predominant use	WAREHOUSE, DISTRIBUTION (407)
Stories	2	Gross sq. ft.	324,190
Building quality	AVERAGE	Net sq. ft.	324,190
Construction class	MASONRY	Heating system	SPACE HEATERS
Building shape	Long Rect or Irreg	Sprinklers	Y
		Elevators	

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$4,718,700 (land) + $12,594,200 (improvements) $17,312,900 (total)	$4,718,700 (land) + $12,594,200 (improvements) $17,312,900 (total)
2008	TAXABLE	NONE OR UNKNOWN	$3,932,300 (land) + $12,564,700 (improvements) $16,497,000 (total)	$3,932,300 (land) + $12,564,700 (improvements) $16,497,000 (total)
2007	TAXABLE	NONE OR UNKNOWN	$3,775,000 (land) + $12,019,900 (improvements) $15,794,900 (total)	$3,775,000 (land) + $12,019,900 (improvements) $15,794,900 (total)

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Assessor Information for parcel number 3523049036

Taxpayer name	LA PIANTA LLC	Parcel number	3523049036
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	352304903607
		Levy code	2360
		Jurisdiction	TUKWILA
		Present use	Industrial(Gen Purpose)
		Appraised value	$1,588,100
Address(es) at this parcel	18400 SOUTHCENTER PKWY 98188		

Legal description

PORTION OF GOV LOT 6 IN SW 1/4 OF SECTION 35-23-04 - BEGIN NE CORNER OF GOV LOT 6 TH N 87-56-03 W 400.6
438.87 FT TO POINT OF BEGINNING TH S 25-01-18 W 439.33 FT TH S 66-36-37 E 486.58 FT TH N 23-28-33 E 431.62 F
269.84 FT TO POINT OF BEGINNING - PARCEL C OF CITY OF TUKWILA BOUNDARY LINE ADJUSTMENT NO L99-0008 I

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number	Recording number
12-29-1994	$29,100,000	LA PIANTA LTD PTNRSHP	SEGALE MARIO A	1410756	199412300689

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Parcel description

Property name	ENTERPRISE RENT A CAR	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Industrial(Gen Purpose)	Plat lot		Access	PUBLIC
Lot area	209,151 sq. ft. (4.80 acres)	Q-S-T-R	SW-35-23-4	Street surface	PAVED

Commercial building description

Building	1 of 1	Building description	SHOP
Year built	1966	Predominant use	INDUSTRIAL LIGHT MANUFACTURING (494)
Stories	1	Gross sq. ft.	1,664
Building quality	AVERAGE	Net sq. ft.	1,664
Construction class	WOOD FRAME	Heating system	FORCED AIR UNIT
Building shape	Rect or Slight Irreg	Sprinklers	N
		Elevators	

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$1,568,600 (land) + $19,500 (improvements) $1,588,100 (total)	$1,568,600 (land) + $19,500 (improvements) $1,588,100 (total)
2008	TAXABLE	NONE OR UNKNOWN	$1,359,400 (land) + $30,400 (improvements) $1,389,800 (total)	$1,359,400 (land) + $30,400 (improvements) $1,389,800 (total)
2007	TAXABLE	NONE OR UNKNOWN	$1,254,000 (land) + $28,800 (improvements)	$1,254,000 (land) + $28,800 (improvements)

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Assessor information for parcel number 3523049017

Taxpayer name	LA PIANTA L L C
Mailing address	PO BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	None

Parcel number	3523049017
Tax Account number	352304901700
Levy code	2300
Jurisdiction	TUKWILA
Present use	Vacant(Industrial)
Appraised value	$653,400

⊟ Legal description

BEG ON ELY MGN OF CO RD IN GL 7 AT A PT 1656.75 FT E & 1048.79 FT N OF SW COR OF SEC TH S 08-13-05 E 36.7
S 73-05-42 E 233.32 FT TH N 16-29-55 E 374 FT TH N 66-02-38 W TO TPOB

⊟ Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number	Recording r
06-09-2008	$0	LA PIANTA L L C	EPC HOLDINGS 763 L L C	2350738	200806170
12-19-2007	$951,000	EPC HOLDINGS 763 LLC	GWI REALTY LLC	2325868	200712200
07-24-2006	$0	GWI REALTY L L C	GACO WESTERN INC	2231307	200608230
06-30-2006	$0	GWI REALTY LLC	GACO WESTERN INC	2220946	200607110

⊟ Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	87,120 sq. ft. (2.00 acres)	Q-S-T-R	SW-35-23-4	Street surface	PAVED

⊟ Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$653,400 (land) + $0 (improvements) $653,400 (total)	$653,400 (land) + $0 (improvements) $653,400 (total)
2008	TAXABLE	NONE OR UNKNOWN	$435,600 (land) + $0 (improvements) $435,600 (total)	$435,600 (land) + $0 (improvements) $435,600 (total)
2007	TAXABLE	NONE OR UNKNOWN	$435,600 (land) + $0 (improvements) $435,600 (total)	$435,600 (land) + $0 (improvements) $435,600 (total)

⊟ Related resources

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Assessor Information for parcel number 3523049041

Taxpayer name	LA PIANTA LLC
Mailing address	P O BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	None

Parcel number	3523049041
Tax Account number	352304904100
Levy code	4300
Jurisdiction	KING COUNTY
Present use	Vacant(Industrial)
Appraised value	$901,500

Legal description

BEG ON ELY MGN OF CO RD AT A PT S 89-03-20 E 1656.75 FT & N 00-56-40 E 1048.79 FT OF SW COR OF SEC TH S 08-13-
N 16-29-55 E 474 FT TH S 64-17-20 E 232 FT M/L TO BANK OF GREEN RIVER TH SLY & ELY ALG SD RIVER BANK TO S LN C
SD ELY MGN TO BEG BEING A POR OF GL 7 & OF SW 1/4 OF SW 1/4 LESS POR OF SD SUBD DAF - BEG SW COR OF SD SE
FRAGER RD TH N 1-54-58 W 165.53 FT ALG SD LN TH ALG C/L OF ESMT FOLG COURSES & DISTS N 88-05-02 E 111 FT TO
54.25 FT TH N 25-55-02 E 13.34 FT TO CURVE TO RGT WITH RAD OF 50 FT THRU C/A OF 83-13-26 ARC DIST OF 72.63 FT T
TPOB TH CONTG S 19-08-28 W 100 FT TH N 70-51-32 W 100 FT TH N 19-08-28 E 100 FT TH S 70-51-32 E 100 FT TO TPOB

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax numbe
05-10-2000	$996,100	LA PIANTA LLC	ROCKWALL INDUSTRIES INC	1752805
10-26-1995	$630,000	ROCKWALL INDUSTRIES INC	METRO LAND DEVELOPMENT INC	1453795
12-27-1994	$0	METRO LAND DVLPMNT INC+SEGALE	LA PIANTA LIMITED PARTNERSHIP	1410451
11-28-1994	$544,300	LA PIANTA LIMITED PTNRSHP	METRO LAND DEVELOPMENT INC	1405749

Parcel description

Property name	VACANT LAND	Plat name		Water system	PRIVATE
Property type	C - COMMERCIAL	Plat block		Sewer system	NONE OR UNKNOWN
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	1,126,897 sq. ft. (25.87 acres)	Q-S-T-R	SE-35-23-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$901,500 (land) + $0 (improvements) $901,500 (total)	$901,500 (land) + $0 (improvements) $901,500 (total)
2008	TAXABLE	NONE OR UNKNOWN	$732,400 (land) + $0 (improvements) $732,400 (total)	$732,400 (land) + $0 (improvements) $732,400 (total)
2007	TAXABLE	NONE OR UNKNOWN	$732,400 (land) + $0 (improvements)	$732,400 (land) + $0 (improvements)

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Assessor Information for parcel number 0222049033

Taxpayer name	LA PIANTA LLC	Parcel number	0222049033
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	022204903300
		Levy code	5025
		Jurisdiction	KING COUNTY
		Present use	Vacant(Industrial)
		Appraised value	$519,900
Address(es) at this parcel	None		

Legal description

PORS OF GLS 10 & 11 IN N 1/2 OF STR 02-22-04 LY E OF E MGN OF FRAGER RD S LESS FOLG BAAP ON N LN OF SD SEC 15-04-10 W 311.46 FT TH S 88-33-10 W 198.17 FT TH N 02-41-45 E 140.14 FT TH N 72-37-15 W TO W MGN OF FRAGER F

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax numb
06-10-2000	$895,100	LA PIANTA LLC	ROCKWALL INDUSTRIES INC	1752605
10-26-1995	$630,000	ROCKWALL INDUSTRIES INC	METRO LAND DEVELOPMENT INC	1453795
12-27-1994	$0	METRO LAND DVLPMNT INC+SEGALE	LA PIANTA LIMITED PARTNERSHIP	1410451
11-28-1994	$544,300	LA PIANTA LIMITED PTNRSHP	METRO LAND DEVELOPMENT INC	1405749

Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	NONE OR UNKNOWN
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	649,915 sq. ft. (14.92 acres)	Q-S-T-R	NW-2-22-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$519,900 (land) + $0 (improvements) $519,900 (total)	$519,900 (land) + $0 (improvements) $519,900 (total)
2008	TAXABLE	NONE OR UNKNOWN	$422,400 (land) + $0 (improvements) $422,400 (total)	$422,400 (land) + $0 (improvements) $422,400 (total)
2007	TAXABLE	NONE OR UNKNOWN	$422,400 (land) + $0 (improvements) $422,400 (total)	$422,400 (land) + $0 (improvements) $422,400 (total)

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Assessor information for parcel number 0222049040

Taxpayer name	LA PIANTA LLC	Parcel number	0222049040
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	022204904001
		Levy code	5025
		Jurisdiction	KING COUNTY
		Present use	Vacant(Industrial)
		Appraised value	$74,500
Address(es) at this parcel	None		

Legal description

POR GL 10-11 BEG 1314.12 FT E FR NW COR OF SEC TH S 77-19-20 E 269.06 FT TH S 15-04-10 W 164.83 FT TO TPO 05-44-15 E 225.46 FT TH S 67-09-20 E 131.54 FT TH S 74-01-20 E 96.81 FT TO TPOB

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax n
05-10-2000	$996,100	LA PIANTA LLC	ROCKWALL INDUSTRIES INC	175260
10-28-1996	$630,000	ROCKWALL INDUSTRIES INC	METRO LAND DEVELOPMENT INC	145379
12-27-1994	$0	METRO LAND DVLPMNT INC+SEGALE	LA PIANTA LIMITED PARTNERSHIP	141045
11-28-1994	$544,300	LA PIANTA LIMITED PTNRSHP	METRO LAND DEVELOPMENT INC	140574

Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	NONE OR UNKNOWN
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	37,296 sq. ft. (0.86 acres)	Q-S-T-R	NW-2-22-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$74,500 (land) + $0 (improvements) $74,500 (total)	$74,500 (land) + $0 (improvements) $74,500 (total)
2008	TAXABLE	NONE OR UNKNOWN	$40,000 (land) + $0 (improvements) $40,000 (total)	$40,000 (land) + $0 (improvements) $40,000 (total)
2007	TAXABLE	NONE OR UNKNOWN	$24,200 (land) + $0 (improvements) $24,200 (total)	$24,200 (land) + $0 (improvements) $24,200 (total)

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Assessor Information for parcel number 0222049043

Taxpayer name	LA PIANTA LLC
Mailing address	PO BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	None

Parcel number	0222049043
Tax Account number	022204904308
Levy code	5025
Jurisdiction	KING COUNTY
Present use	Vacant(Industrial)
Appraised value	$1,811,200

Legal description

POR GL 10 DAF BAAP ON N LN GL 10 835.00 FT E FR NW COR SD SEC TH S 42-48-17 E 165.27 FT TO TPOB TH S 57-
S 45-27-10 E 216.97 FT TH S 31-12-21 W 731.63 FT TH S 88-43-44 E 896.82 FT TO WLY MGN MESS CO RD NO 76 (FF
TO TPOB

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number	Record
02-28-1996	$4,311,396	LA PIANTA LIMITED PARTNERSHIP	SEGALE M A INC	1472208	1996(
02-28-1996	$0	LA PIANTA LIMITED PARTNERSHIP	M A SEGALE INC	1918158	200210
01-07-1986	$0	ZORAGGEN FAMILY LIMITED PARTNE	ZORAGGEN ROSE	870594	1986(

Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	905,612 sq. ft. (20.79 acres)	Q-S-T-R	NW-2-22-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$1,811,200 (land) + $0 (improvements) $1,811,200 (total)	$1,811,200 (land) + $0 (improvements) $1,811,200 (total)
2008	TAXABLE	NONE OR UNKNOWN	$1,584,800 (land) + $0 (improvements) $1,584,800 (total)	$1,584,800 (land) + $0 (improvements) $1,584,800 (total)
2007	TAXABLE	NONE OR UNKNOWN	$1,584,800 (land) + $0 (improvements) $1,584,800 (total)	$1,584,800 (land) + $0 (improvements) $1,584,800 (total)

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Assessor Information for parcel number 0222049057

Taxpayer name	**LA PIANTA LLC**
Mailing address	**PO BOX 88028 TUKWILA WA 98138**
Address(es) at this parcel	**None**

Parcel number	**0222049057**
Tax Account number	**022204905701**
Levy code	**5025**
Jurisdiction	**KING COUNTY**
Present use	**Vacant(Industrial)**
Appraised value	**$133,800**

⊟ Legal description

POR OF GL 10 IN NW 1/4 OF NW 1/4 STR 02-22-04 DAF-BEG NW COR OF SD SEC 02 TH S 01-55-10 E ALG W LN OF 731.63 FT TH S 88-45-19 E 896.82 FT TO W MGN OF FRAGER RD TH SLY ALG SD W MGN TO S LN OF SD NW 1/4 OF LN TO TPOB LESS BAAP S 01-55-10 E ON W LN OF SD SUBD 812.83 FT FR NW COR OF SD SEC 02 TH N 37-42-25 I W 104.89 FT TO W LN OF SD SUBD TH N 1-55-10 W ALG SD W LN 150.65 FT TO TPOB LESS POR OF GL 10 LY WIT SD SEC 02 TH S 36-52-33 W 324.55 FT TH S 11-26-35 W 51.27 FT TH S 29-00-00 E 99.12 FT TH N 78-09-50 E 236.0 HWY

⊟ Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax
05-10-2000	$990,900	LA PIANTA LLC	ROCKWALL INDUSTRIES INC	17526
10-26-1995	$625,000	ROCKWALL INDUSTRIES INC	M A SEAGLE INC	14537
12-27-1994	$0	M A SEGALE INC	LA PIANTA LIMITED PARTNERSHIP	14104
11-28-1994	$553,600	LA PIANTA LIMITED PTNRSHP	M A SEGALE INC	14057
01-07-1986	$0	ZGRAGGEN FAMILY LIMITED PARTNE	ZGRAGGEN ROSE	87058

⊟ Parcel description

Property name	**VACANT LAND**	Plat name		Water system	**WATER DISTRICT**
Property type	**C - COMMERCIAL**	Plat block		Sewer system	**PUBLIC**
Present use	**Vacant(Industrial)**	Plat lot		Access	**PUBLIC**
Lot area	**382,456 sq. ft. (8.78 acres)**	Q-S-T-R	**NW-2-22-4**	Street surface	**PAVED**

⊟ Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$133,800 (land) + $0 (improvements) $133,800 (total)	$133,800 (land) + $0 (improvements) $133,800 (total)
2008	TAXABLE	NONE OR UNKNOWN	$87,900 (land) + $0 (improvements) $87,900 (total)	$87,900 (land) + $0 (improvements) $87,900 (total)
2007	TAXABLE	NONE OR UNKNOWN	$15,400 (land)	$15,400 (land)

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Assessor Information for parcel number 0222049011

Taxpayer name	LA PIANTA LLC
Mailing address	PO BOX 88028 TUKWILA WA 98138
Address(es) at this parcel	None

Parcel number	0222049011
Tax Account number	022204901106
Levy code	5025
Jurisdiction	KING COUNTY
Present use	Vacant(Industrial)
Appraised value	$882,700

Legal description

PORTION OF FOLLOWING PARCEL "A" LOCATED WITHIN GOVT LOT 9 IN SW QTR NW QTR STR 02-22-04 PARCEL "A
STR 02-22-04 DAF: BEGINNING AT POINT ON W MARGIN OF FRAGER RD AT INTERSECTION WITH S LINE OF PARCEL (
7611170105 TH N88-43-44W ALONG S LINE OF SAID SEGALE PARCEL 896.82 FT TO SW CORNER OF SAID PARCEL T
IN SAID WLY LINE TH N46-27-10W ALONG SAID WLY LINE & NWLY EXTENSION OF SAID LINE TO SELY LINE OF PARC
WLY & SWLY ALONG SAID SELY LINE & SELY LINE OF INTERSTATE HWY NO 5 TO S LINE OF NE QTR NE QTR SAID SE
CONVEYED TO SEATTLE & WALLA WALLA RR & TRANSPORTATION CO BY DEED RECORDED IN VOL 2, PAGE 42 OF I
200TH ST TH EAST ALONG N LINE OF SAID S 200TH ST TO INTERSECTION WITH W LINE OF FRAGER RD TH NLY ALON
SAID SW QTR NW QTR STR 03-22-04 LYING EAST OF FRAGER RD & WEST OF W BANK OF GREEN RIVER EXC PORTIC
AND E HALF NE QTR STR 03-22-04 DAF: BEGINNING AT NW CORNER SAID SEC 2 TH S01-09-08E 1392.41 FT ALONG V
M.A. SEGALE PROPERTY AS SHOWN ON SURVEY UNDER RECORDING NO 7707280568 TH S11-41-30W 352.30 FT TH
N36-52-33E 324.55 FT TO TPOB & EXC PORTIONS THEREOF WITHIN TRACTS CONVEYED TO CITY OF KENT BY DEED
TO CITY OF KENT BY DEED UNDER RECORDING NO 9705281238

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax number
02-22-2001	$0	KING COUNTY	KENT CITY OF	1938071
05-10-2000	$990,900	LA PIANTA LLC	ROCKWALL INDUSTRIES INC	1752604
04-22-1997	$0	KENT CITY OF	ROCKWALL INDUSTRIES INC	1545862
04-22-1997	$0	KENT CITY OF	ROCKWALL INDUSTRIES INC	1545854
10-26-1995	$625,000	ROCKWALL INDUSTRIES INC	M A SEAGLE INC	1453796
12-27-1994	$0	M A SEGALE INC	LA PIANTA LIMITED PARTNERSHIP	1410440
11-28-1994	$563,600	LA PIANTA LIMITED PTNRSHP	M A SEGALE INC	1406752

Parcel description

Property name	VACANT LAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	1,176,991 sq. ft. (27.02 acres)	Q-S-T-R	NW-2-22-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$882,700 (land) + $0 (improvements) $882,700 (total)	$882,700 (land) + $0 (improvements) $882,700 (total)
2008	TAXABLE	NONE OR UNKNOWN	$588,400 (land)	$588,400 (land)

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- 122° 19' 52.90"

Assessor Information for parcel number 0222049015

Taxpayer name	LA PIANTA LLC	Parcel number	0222049015
Mailing address	PO BOX 88028 TUKWILA WA 98138	Tax Account number	022204901502
		Levy code	5025
		Jurisdiction	KING COUNTY
		Present use	Vacant(Industrial)
		Appraised value	$1,584,100
Address(es) at this parcel	None		

Legal description

GOVT LOT 8 IN SW QTR STR 02-22-04 EXC S 40 FT THEREOF WITHIN S 204TH ST CONVEYED TO KING CO BY DEED U
CONVEYED TO KING CO BY DEED UNDER RECORDING NO 1731274 & EXC PORTION WITHIN FRAGER RD GRANTED A
& EXC PORTIONS CONDEMNED BY DRAINAGE DIST NO 2 IN KING CO SUP CT CAUSE NO 47032; TGW PORTION OF NE
QTR SEC 3 (BEING ALSO NW CORNER GOVT LOT 8 STR 02-22-04) TH S00-30-45E ALONG E LINE OF SAID SE QTR SE
SOUTH OF & PARALLEL TO EAST-WEST CENTERLINE SAID SEC 3 DISTANCE OF 711.96 FT TH S00-25-36E 1264.25 F
N89-40-00E ALONG LINE 40 FT NORTH OF & PARALLEL TO S LINE OF SAID NE QTR SE QTR 713.84 FT TO E LINE SAID
TPOB EXC THOSE PORTIONS CONDEMNED BY DRAINAGE DIST NO 2 IN KING CO SUP CT CAUSE NO 47302 & EXC PO
UNDER RECORDING NO 9705231403

Sales/Quit Claims/Transfers

Sale date	Sale price	Buyer	Seller	Excise tax numb
04-22-1997	$0	KENT CITY OF	LA PIANTA LIMITED PARTNERSHIP	1544869
12-29-1994	$1,769,000	LA PIANTA LIMITED PARTNERSHIP	M A SEGALE INC	1410769

Parcel description

Property name	VACANT - WETLAND	Plat name		Water system	WATER DISTRICT
Property type	C - COMMERCIAL	Plat block		Sewer system	PUBLIC
Present use	Vacant(Industrial)	Plat lot		Access	PUBLIC
Lot area	2,112,224 sq. ft. (48.49 acres)	Q-S-T-R	SW-2-22-4	Street surface	PAVED

Taxable value history

Tax year	Tax status	Taxable value reason	Appraised value	Taxable value
2009	TAXABLE	NONE OR UNKNOWN	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)
2008	TAXABLE	NONE OR UNKNOWN	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)
2007	TAXABLE	NONE OR UNKNOWN	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)	$1,584,100 (land) + $0 (improvements) $1,584,100 (total)

Related resources

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Washington, DC 20314-1000

CECW-EG

Manual
No. 1110-2-1913

30 April 2000

Engineering and Design
DESIGN AND CONSTRUCTION OF LEVEES

1. Purpose. The purpose of this manual is to present basic principles used in the design and construction of earth levees.

2. Applicability. This manual applies to all Corps of Engineers Divisions and Districts having responsibility for the design and construction of levees.

3. Distribution. This manual is approved for public release; distribution is unlimited.

4. General. This manual is intended as a guide for designing and constructing levees and not intended to replace the judgment of the design engineer on a particular project.

FOR THE COMMANDER:

RUSSELL L. FUHRMAN
Major General, USA
Chief of Staff

This manual supersedes EM 1110-2-1913, dated 31 March 1978.

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Design and Construction of Levees

ENGINEER MANUAL

AVAILABILITY

Electronic copies of this and other U.S. Army Corps of Engineers (USACE) publications are available on the Internet at http://www.usace.army.mil/inet/usace-docs/. This site is the only repository for all official USACE engineer regulations, circulars, manuals, and other documents originating from HQUSACE. Publications are provided in portable document format (PDF).

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U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-1913

Manual
No. 1110-2-1913

30 April 2000

Engineering and Design
DESIGN AND CONSTRUCTION OF LEVEES

Table of Contents

Appendix H
Notation

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List of Figures

Chapter 1
Introduction

1-1. Purpose

The purpose of this manual is to present basic principles used in the design and construction of earth levees.

1-2. Applicability

This manual applies to all Corps of Engineers Divisions and Districts having responsibility for designing and constructing levees.

1-3. References

Appendix A contains a list of required and related publications pertaining to this manual. Unless otherwise noted, all references are available on interlibrary loan from the Research Library, ATTN: CEWES-IM-MI-R, U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

1-4. Objective

The objective of this manual is to develop a guide for design and construction of levees. The manual is general in nature and not intended to supplant the judgment of the design engineer on a particular project.

1-5. General Considerations

a. General

(1) The term levee as used herein is defined as an embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein.

(2) Even though levees are similar to small earth dams they differ from earth dams in the following important respects: (a) a levee embankment may become saturated for only a short period of time beyond the limit of capillary saturation, (b) levee alignment is dictated primarily by flood protection requirements, which often results in construction on poor foundations, and (c) borrow is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal. Selection of the levee section is often based on the properties of the poorest material that must be used.

(3) Numerous factors must be considered in levee design. These factors may vary from project to project, and no specific step-by step procedure covering details of a particular project can be established. However, it is possible to present general, logical steps based on successful past projects that can be followed in levee design and can be used as a base for developing more specific procedures for any particular project. Such a procedure is given in Table 1-1. Information for implementing this procedure is presented in subsequent chapters.

Table 1-1
Major and Minimum Requirements

Step	Procedure
1	Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	Initiate final exploration to provide: a. Additional information on soil profiles. b. Undisturbed strengths of foundation materials. c. More detailed information on borrow areas and other required excavations.
4	Using the information obtained in Step 3: a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: a. Underseepage and through seepage. b. Slope stability. c. Settlement. d. Trafficability of the levee surface.
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final quantities needed; determine final borrow area locations.
10	Design embankment slope protection.

(4) The method of construction must also be considered. In the past levees have been built by methods of compaction varying from none to carefully controlled compaction. The local economic situation also affects the selection of a levee section. Traditionally, in areas of high property values, high land use, and good foundation conditions, levees have been built with relatively steep slopes using controlled compaction, while in areas of lower property values, poor foundations, or high rainfall during the construction season, uncompacted or semicompacted levees with flatter slopes are more typical. This is evident by comparing the steep slopes of levees along the industrialized Ohio River Valley with levees along the Lower Mississippi River which have much broader sections with gentler slopes. Levees built with smaller sections and steeper slopes generally require more comprehensive investigation and analysis than do levees with broad sections and flatter slopes whose design is more empirical. Where rainfall and foundation conditions permit, the trend in design of levees is toward sections with steeper slopes. Levee maintenance is another factor that often has considerable influence on the selection of a levee section.

b. Levee types according to location. Levees are broadly classified according to the area they protect as either urban or agricultural levees because of different requirements for each. As used in this manual, urban and agricultural levees are defined as follows:

(1) Urban levees. Levees that provide protection from flooding in communities, including their industrial, commercial, and residential facilities.

(2) Agricultural levees. Levees that provide protection from flooding in lands used for agricultural purposes.

c. Levee types according to use. Some of the more common terms used for levees serving a specific purpose in connection with their overall purpose of flood protection are given in Table 1-2.

Table 1-2
Classification of Levees According to Use

Type	Definition
Mainline and tributary levees	Levees that lie along a mainstream and its tributaries, respectively.
Ring levees	Levees that completely encircle or "ring" an area subject to inundation from all directions.
Setback levees	Levees that are built landward of existing levees, usually because the existing levees have suffered distress or are in some way being endangered, as by river migration.
Sublevees	Levees built for the purpose of underseepage control. Sublevees encircle areas behind the main levee which are subject, during high-water stages, to high uplift pressures and possibly the development of sand boils. They normally tie into the main levee, thus providing a basin that can be flooded during high-water stages, thereby counterbalancing excess head beneath the top stratum within the basin. Sublevees are rarely employed as the use of relief wells or seepage berms make them unnecessary except in emergencies.
Spur levees	Levees that project from the main levee and serve to protect the main levee from the erosive action of stream currents. Spur levees are not true levees but training dikes.

d. Causes of Levee Failures. The principal causes of levee failure are

(1) Overtopping.

(2) Surface erosion.

(3) Internal erosion (piping).

(4) Slides within the levee embankment or the foundation soils.

Chapter 2
Field Investigations

2-1. Preliminary and Final Stage

Many field investigations are conducted in two stages: a preliminary stage and a final (design) stage. Normally, a field investigation in the preliminary stage is not extensive since its purpose is simply to provide general information for project feasibility studies. It will usually consist of a general geological reconnaissance with only limited subsurface exploration and simple soil tests. In the design stage, more comprehensive exploration is usually necessary, with more extensive geological reconnaissance, borings, test pits, and possibly geophysical studies. The extent of the field investigation depends on several factors. Table 2-1 lists these factors together with conditions requiring extensive field investigations and design studies. Sometimes field tests such as vane shear tests, groundwater observations, and field pumping tests are necessary. Table 2-2 summarizes, in general, the broad features of geologic and subsurface investigations.

Section I
Geological Study

2-2. Scope

A geological study usually consists of an office review of all available geological information on the area of interest and an on-site (field) survey. Since most levees are located in alluvial floodplains, the distribution and engineering characteristics of alluvial deposits in the vicinity of proposed levees must be evaluated. The general distribution, nature, and types of floodplain deposits are directly related to changes in the depositional environment of the river and its tributaries. Each local area in the floodplain bears traces of river action, and the alluvial deposits there may vary widely from those in adjacent areas. The general nature and distribution of sediments can be determined through a study of the pattern of local river changes as a basis for selection of boring locations.

Table 2-1
Factors Requiring Intensive Field Investigations and Design Studies

Factor	Field Investigations and Design Studies Should be more Extensive Where:
Previous experience	There is little or no previous experience in the area particularly with respect to levee performance
Consequences of failure	Consequences of failure involving life and property are great (urban areas for instance)
Levee height	Levee heights exceed 3 m (10 ft)
Foundation conditions	Foundation soils are weak and compressible
	Foundation soils are highly variable along the alignment
	Potential underseepage problems are severe
	Foundation sands may be liquefaction susceptible
Duration of high water	High water levels against the levee exist over relatively long periods
Borrow materials	Available borrow is of low quality, water contents are high, or borrow materials are variable along the alignment
Structure in levees	Reaches of levees are adjacent to concrete structures

Table 2-2
Stages of Field Investigations

1. Investigation or analysis produced by field reconnaissance and discussion with knowledgeable people is adequate for design where:

   a. Levees are 3 m (10 ft) or less in height.

   b. Experience has shown foundations to be stable and presenting no underseepage problems.

   Use standard levee section developed through experience.

2. *Preliminary geological investigation*:

   a. Office *study*: Collection and study of

      (1) Topographic, soil, and geological maps.
      (2) Aerial photographs.
      (3) Boring logs and well data.
      (4) Information on existing engineering projects.

   b. *Field survey*: Observations and geology of area, documented by written notes and photographs, including such features as:

      (1) Riverbank slopes, rock outcrops, earth and rock cuts or fills.
      (2) Surface materials.
      (3) Poorly drained areas.
      (4) Evidence of instability of foundations and slopes.
      (5) Emerging seepage.
      (6) Natural and man-made physiographic features.

3. *Subsurface exploration and field testing and more detailed geologic study*: Required for all cases except those in 1 above. Use to decide the need for and scope of subsurface exploration and field testing:

   a. *Preliminary phase*:

      (1) Widely but not necessarily uniformly spaced disturbed sample borings (may include split-spoon penetration tests).
      (2) Test pits excavated by backhoes, dozers, or farm tractors.
      (3) Geophysical surveys (e.g., seismic or electrical resistivity) or cone penetrometer test to interpolate between widely spaced borings.
      (4) Borehole geophysical tests.

   b. *Final phase*:

      (1) Additional disturbed sample borings.
      (2) Undisturbed sample borings.
      (3) Field vane shear tests for special purposes.
      (4) Field pumping tests (primarily in vicinity of structures).
      (5) Water table observations (using piezometers) in foundations and borrow areas.

2-3. Office Study

The office study begins with a search of available information, such as topographic, soil, and geological maps and aerial photographs. Pertinent information on existing construction in the area should be obtained. This includes design, construction, and performance data on utilities, highways, railroads, and hydraulic structures. Available boring logs should be secured. Federal, state, county, and local agencies and private organizations should be contacted for information. The GIS (Geographic Information System) became used extensively in major range of projects. It is capable of compiling large multi-layered data bases, interactively analyzing and manipulating those data bases, and generating and displaying resultant thematic maps and statistics to aid in engineering management decisions. Federal, state, and private organizations provide free internet access to such systems. Table 2-3 shows some of the contour maps GIS systems provide.

Table 2-3
Types of Contour Maps

Contour Type	Uses	
Geologic Structure Elevation Maps	Contour maps in which each line represents the elevation of the top of a geological material or facies	GIS can produce these maps based on the selection of one of four structure parameters
Geologic formations	Contours the top of a user-defined geologic formation	
Blow counts	Contours the top of a structure identified by the first, second, or third occurrence of a specified range of blow counts	A blow count is defined as the number of standard blows required to advance a sampling device into 150 mm (6 in.) of soil
Soil units	Contours the top of a structure identified by the first, second, or third occurrence of one or more soil types	
Fluid level elevation - water table contour maps	Show elevation data (hydraulic head) from unconfined water bearing units where the fluid surface is in equilibrium with atmospheric pressure	Help to evaluate the direction of ground water flow and the energy gradient under which it is flowing
Fluid level elevation - potentiometric surface maps	Show elevation data from confined water bearing units where the fluid surface is under pressure because of the presence of a confining geologic unit	
Hydraulic conductivity	Show the rate of water flow through soil under a unit gradient per unit area	GIS stores vertical and horizontal conductivity data for up to five water bearing zones
	Portray the variations in the water-bearing properties of materials which comprise each water bearing zone	Necessary parameter for computing ground water flow rates, which is important since groundwater velocity exerts a major control on plume shape

2-4. Field Survey

The field survey is commenced after becoming familiar with the area through the office study. Walking the proposed alignment and visiting proposed borrow areas are always an excellent means of obtaining useful information. Physical features to be observed are listed in Table 2-2. These items and any others of significance should be documented by detailed notes, supplemented by photographs. Local people or organizations having knowledge of foundation conditions in the area should be interviewed.

2-5. Report

When all available information has been gathered and assimilated, a report should be written that in essence constitutes a geological, foundation, and materials evaluation report for the proposed levee. All significant factors that might affect the alignment and/or design should be clearly pointed out and any desirable changes in alignment suggested. All maps should be to the same scale, and overlays of maps, e.g., topography and soil type, aerial photograph and topography, etc., to facilitate information correlation is desirable. The development of a project GIS will simplify and expedite consistently georeferenced map products.

Section II
Subsurface Exploration

2-6. General

a. Because preliminary field investigations usually involve only limited subsurface exploration, only portions of the following discussion may be applicable to the preliminary stage, depending on the nature of the project.

b. The subsurface exploration for the design stage generally is accomplished in two phases, which may be separate in sequence, or concurrent: (1) Phase 1, the main purpose of which is to better define the geology of the area, the soil types present and to develop general ideas of soil strengths and permeabilities; (2) Phase 2, provides additional information on soil types present and usually includes the taking of undisturbed samples for testing purposes.

2-7. Phase 1 Exploration

Phase 1 exploration consists almost entirely of disturbed sample borings and perhaps test pits excavated with backhoes, dozers, farm tractors, etc., as summarized in Table 2-4, but may also include geophysical surveys which are discussed later.

Table 2-4
Phase I Boring and Sampling Techniques

Technique	Remarks
1. Disturbed sample borings	
a. Split-spoon or standard penetration test	1-a. Primarily for soil identification but permits estimate of shear strength of clays and crude estimate of density of sands; see paragraph 5-3d of EM 1110-1-1906
	Preferred for general exploration of levee foundations; indicates need and locations for undisturbed samples
b. Auger borings	1-b. Bag and jar samples can be obtained for testing
2. Test pits	2. Use backhoes, dozers, and farm tractors
3. Trenches	3. Occasionally useful in borrow areas and levee foundations

2-8. Phase 2 Exploration

Phase 2 subsurface exploration consists of both disturbed and undisturbed sample borings and also may include geophysical methods. Undisturbed samples for testing purposes are sometimes obtained by handcarving block samples from test pits but more usually by rotary and push-type drilling methods (using samplers such as the Denison sampler in extremely hard soils or the thin-walled Shelby tube fixed piston sampler in most soils). Samples for determining consolidation and shear strength characteristics and values of density and permeability should be obtained using undisturbed borings in which 127-mm- (5-in.-) diameter samples are taken in cohesive materials and 76.2-mm- (3-in.-) diameter samples are taken in cohesionless materials. EM 1110-1-1906 gives details of drilling and sampling techniques.

2-9. Borings

a. Location and spacing. The spacing of borings and test pits in Phase 1 is based on examination of airphotos and geological conditions determined in the preliminary stage or known from prior experience in the area, and by the nature of the project. Initial spacing of borings usually varies from 60 to 300 m (nominally 200 to 1,000 ft) along the alignment, being closer spaced in expected problem areas and wider spaced in nonproblem areas. The spacing of borings should not be arbitrarily uniform but rather should be based on available geologic information. Borings are normally laid out along the levee centerline but can be staggered along the alignment in order to cover more area and to provide some data on nearby borrow materials. At least one boring should be located at every major structure during Phase 1. In Phase 2, the locations of additional general sample borings are selected based on Phase 1 results. Undisturbed sample borings are located where data on soil shear strength are most needed. The best procedure is to group the foundation profiles developed on the basis of geological studies and exploration into reaches of similar conditions and then locate undisturbed sample borings so as to define soil properties in critical reaches.

b. Depth. Depth of borings along the alignment should be at least equal to the height of proposed levee at its highest point but not less than 3 m (nominally 10 ft). Boring depths should always be deep enough to provide data for stability analyses of the levee and foundation. This is especially important when the levee is located near the riverbank where borings must provide data for stability analyses involving both levee foundation and riverbank. Where pervious or soft materials are encountered, borings should extend through the permeable material to impervious material or through the soft material to firm material. Borings at structure locations should extend well below invert or foundation elevations and below the zone of significant influence created by the load. The borings must be deep enough to permit analysis of approach and exit channel stability and of underseepage conditions at the structure. In borrow areas, the depth of exploration should extend several feet below the practicable or allowable borrow depth or to the ground-water table. If borrow is to be obtained from below the groundwater table by dredging or other means, borings should be at least 3 m (nominally 10 ft) below the bottom of the proposed excavation.

2-10. Geophysical Exploration

a. It is important to understand the capabilities of the different geophysical methods, so that they may be used to full advantage for subsurface investigations. Table 2-5 summarizes those geophysical methods most appropriate to levee exploration. These methods are a fairly inexpensive means of exploration and are very useful for correlating information between borings which, for reasons of economy, are spaced at fairly wide intervals. Geophysical data must be interpreted in conjunction with borings and by qualified, experienced personnel. Because there have been significant improvements in geophysical instrumentation and interpretation techniques in recent years, more consideration should be given to their use.

b. Currently available geophysical methods can be broadly subdivided into two classes: those accomplished entirely from the ground surface and those which are accomplished from subsurface borings. Applicable geophysical ground surface exploration methods include: (1) seismic methods, (2) electrical resistivity, (3) natural potential (SP) methods, (4) electromagnetic induction methods, and (5) ground penetrating radar. Information obtained from seismic surveys includes material velocities, delineation of interfaces between zones of differing velocities, and the depths to these interfaces. The electrical resistivity survey is used to locate and define zones of different electrical properties such as pervious and impervious zones or zones of low resistivity such as clayey strata. Both methods require differences in properties of levee and/or foundation materials in order to be effective. The resistivity method requires a resistivity contrast between materials being located, while the seismic method requires contrast in wave transmission velocities. Furthermore, the seismic refraction method requires that any underlying stratum transmit waves

Table 2-5
Applicable Geophysical Methods of Exploration[a]

	Top of Bedrock	Fault Detection	Suspected Voids or Cavity Detection	In Situ Elastic Moduli (Velocities)	Material Boundaries, Dip, ...	Subsurface Conduits and Vessels	Landfill Boundaries
Seismic Refraction	W	S		W	S		
Seismic Reflection	S	S	S		W		
Natural Potential (SP)						S	
DC Resistivity	S	S	S		S	S	W
Electro-Magnetics		S			S	W	W
Ground Penetrating Radar	S	S	S		S	S	S
Gravity		S	S		S		
Magnetics		S					S

W - works well in most materials and natural configurations.
S - works under special circumstances of favorable materials or configurations.
Blank - not recommended.
[a] After EM 1110-1-1802.

at a higher velocity than the overlying stratum. Difficulties arise in the use of the seismic method if the surface terrain and/or layer interfaces are steeply sloping or irregular instead of relatively horizontal and smooth. Therefore, in order to use these methods, one must be fully aware of what they can and cannot do. EM 1110-1-1802 describes the use of both seismic refraction and electrical resistivity. Telford et al. (1990) is a valuable, gėneral text on geophysical exploration. Applicable geophysical exploration methods based on operation from the ground surface are summarized in Table 2-5. A resistivity survey measures variations in potential of an electrical field within the earth by a surface applied current. Variation of resistivity with depth is studied by changing electrode spacing. The data is then interpreted as electrical resistivity expressed as a function of depth. (Telford et al. 1990; EM 1110-1-1802)

c. Downhole geophysical logging can be used with success in correlating subsurface soil and rock stratification and in providing quantitative engineering parameters such as porosity, density, water content, and moduli. They also provide valuable data for interpreting surface geophysical data. The purpose in using these methods is not only to allow cost savings, but the speed, efficiency and often much more reliable information without lessening the quality of the information obtained. Electromagnetic (EM) induction surveys use EM transmitters that generate currents in subsurface materials. These currents produce secondary magnetic fields detectable at the surface. Simple interpretation techniques are advantages of these methods, making EM induction techniques particularly suitable for horizontal profiling. EM horizontal profiling surveys are useful for detecting anomalous conditions along the centerline of proposed levee construction or along existing levees. Self potential (SP) methods are based on change of potential of ground by human action or alteration of original condition. Four electric potentials due to fluid flow, electrokinetic or streaming, liquid junction or diffusion, mineralizaion, and solution differing concentration, are known.

The qualitative application of this method is relatively simple and serves best for detection of anomalous seepage through, under, or around levees (Butler and Llopis, 19909; EM 1110-1-1802).

Section III
Field Testing

2-11. Preliminary Strength Estimates

It is often desirable to estimate foundation strengths during Phase 1 of the exploration program. Various methods of preliminary appraisal are listed in Table 2-6.

Table 2-6
Preliminary Appraisal of Foundation Strengths

Method	Remarks
1. Split-spoon penetration resistance	1-a. Unconfined compressive strength in hundreds kPa (or tons per square foot), of clay is about 1/8 of number of blows per 0.3 m (1 ft), or N/8, but considerable scatter must be expected. Generally not helpful where N is low
	1-b. In sands, N values less than about 15 indicate low relative densities. N values should not be used to estimate relative densities for earthquake design
2. Natural water content of disturbed or general type samples	2. Useful when considered with soil classification, and previous experience is available
3. Hand examination of disturbed samples	3. Useful where experienced personnel are available who are skilled in estimating soil shear strengths
4. Position of natural water contents relative to liquid and plastic limits	4-a. Useful where previous experience is available
	4-b. If natural water content is close to plastic limit foundation shear strength should be high
	4-c. Natural water contents near liquid limit indicate sensitive soil usually with low shear strengths
5. Torvane or pocket penetrometer tests on intact portions of general samples or on walls of test trenches	5. Easily performed and inexpensive but may underestimate actual values ; useful only for preliminary strength classifications

2-12. Vane Shear Tests

Where undisturbed samples are not being obtained or where samples of acceptable quality are difficult to obtain, in situ vane shear tests may be utilized as a means of obtaining undrained shear strength. The apparatus and procedure for performing this test are described in ASTM D 2573. The results from this test may be greatly in error where shells or fibrous organic material are present. Also, test results in high plasticity clays must be corrected using empirical correction factors as given by Bjerrum (1972) (but these are not always conservative).

2-13. Groundwater and Pore Pressure Observations

Piezometers to observe groundwater fluctuations are rarely installed solely for design purposes but should always be installed in areas of potential underseepage problems. The use and installation of piezometers are described in EM 1110-2-1908. Permeability tests should always be made after installation of the piezometers; these tests provide information on foundation permeability and show if piezometers are functioning. Testing and interpretation procedures are described in EM 1110-2-1908.

2-14. Field Pumping Tests

The permeability of pervious foundation materials can often be estimated with sufficient accuracy by using existing correlations with grain-size determination; see TM 5-818-5. However, field pumping tests are the most accurate means of determining permeabilities of stratified in situ deposits. Field pumping tests are expensive and usually justified only at sites of important structures and where extensive pressure relief well installations are planned. The general procedure is to install a well and piezometers at various distances from the well to monitor the resulting drawdown during pumping of the well. Appendix III of TM 5-818-5 gives procedures for performing field pumping tests.

Chapter 3
Laboratory Testing

3-1. General

a. Reference should be made to EM 1110-1-1906 for current soil testing procedures, and to EM 1110-2-1902 for applicability of the various shear strength tests in stability analyses.

b. Laboratory testing programs for levees will vary from minimal to extensive, depending on the nature and importance of the project and on the foundation conditions, how well they are known, and whether existing experience and correlations are applicable. Since shear and other tests to determine the engineering properties of soils are expensive and time-consuming, testing programs generally consist of water content and identification tests on most samples and shear, consolidation, and compaction tests only on representative samples of foundation and borrow materials. It is imperative to use all available data such as geological and geophysical studies, when selecting representative samples for testing. Soil tests that may be included in laboratory testing programs are listed in Table 3-1 for fine-grained cohesive soils and in Table 3-2 for pervious soils, together with pertinent remarks on purposes and scope of testing.

Table 3-1
Laboratory Testing of Fine-Grained Cohesive Soils

Test	Remarks
Visual classification and water content determinations	On all samples
Atterberg limits	On representative samples of foundation deposits for correlation with shear or consolidation parameters, and borrow soils for comparison with natural water contents, or correlations with optimum water content and maximum densities
Permeability	Not required; soils can be assumed to be essentially impervious in seepage analyses
Consolidation	Generally performed on undisturbed foundation samples only where:
	a. Foundation clays are highly compressible
	b. Foundations under high levees are somewhat compressible
	c. Settlement of structures within levee systems must be accurately estimated
	Not generally performed on levee fill; instead use allowances for settlement within levees based on type of compaction. Sometimes satisfactory correlations of Atterberg limits with coefficient of consolidation can be used. Compression index can usually be estimated from water content.
Compaction	*a.* Required only for compacted or semi-compacted levees
	b. Where embankment is to be fully compacted, perform standard 25-blow compaction tests
	c. Where embankment is to be semi-compacted, perform 15-blow compaction tests
Shear strength	*a.* Unconfined compression tests on saturated foundation clays without joints or slickensides
	b. Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability
	c. R triaxial and S direct shear: Generally required only when levees are high and/or foundations are weak, or at locations where structures exist in levees
	d. Q, R, and S tests on fill materials compacted at appropriate water contents to densities resulting from the expected field compaction effort

Table 3-2
Laboratory Testing of Pervious Materials

Test	Remarks
Visual classification	Of all jar samples
In situ density determinations	Of Shelby-tube samples of foundation sands where liquefaction susceptibility must be evaluated
Relative density	Maximum and minimum density tests should be performed in seismically active areas to determine in situ relative densities of foundation sands and to establish density control of sand fills
Gradation	On representative foundation sands:
	a. For correlating grain-size parameters with permeability or shear strength
	b. For size and distribution classifications pertinent to liquefaction potential
Permeability	Not usually performed. Correlations of grain-size parameters with permeability or shear strength used. Where underseepage problems are serious, best guidance obtained by field pumping tests
Consolidation	Not usually necessary as consolidation under load is insignificant and occurs rapidly
Shear strength	For loading conditions other than dynamic, drained shear strength is appropriate. Conservative values of φ' can be assumed based on S tests on similar soils. In seismically active areas, cyclic triaxial tests may be performed

3-2. Classification and Water Content Determinations

After soil samples have been obtained in subsurface exploration of levee foundations and borrow areas, the first and essential step is to make visual classifications and water content determinations on all samples (except that water content determinations should not be made on clean sands and gravels). These samples may be jar or bag samples obtained from test pits, disturbed or undisturbed drive samples, or auger samples. Field descriptions, laboratory classifications, and water content values are used in preparing graphic representations of boring logs. After examining these data, samples of fine-grained soils are selected for Atterberg limits tests, and samples of coarse-grained soils for gradation tests.

Section I
Fine-Grained Soils

3-3. Use of Correlations

Comparisons of Atterberg limits values with natural water contents of foundation soils and use of the plasticity chart itself (Figure 3-1), together with split-spoon driving resistance, geological studies, and previous experience often will indicate potentially weak and compressible fine-grained foundation strata and thus the need for shear and perhaps consolidation tests. In some cases, in the design of low levees on familiar foundation deposits for example, correlations between Atterberg limits values and consolidation or shear strength characteristics may be all that is necessary to evaluate these characteristics. Examples of correlations among Atterberg limits values, natural water content, shear strength and consolidation characteristics are shown in Figures 3-2 and 3-3. Correlations based on local soil types and which distinguish between normally and overconsolidated conditions are preferable. Such correlations may also be used to reduce the number of tests required for design of higher levees. As optimum water content may in some cases be correlated with Atterberg limits, comparisons of Atterberg limits and natural water contents of borrow soils as shown in Figure 3-4 can indicate whether the borrow materials are suitable for obtaining adequate compaction.

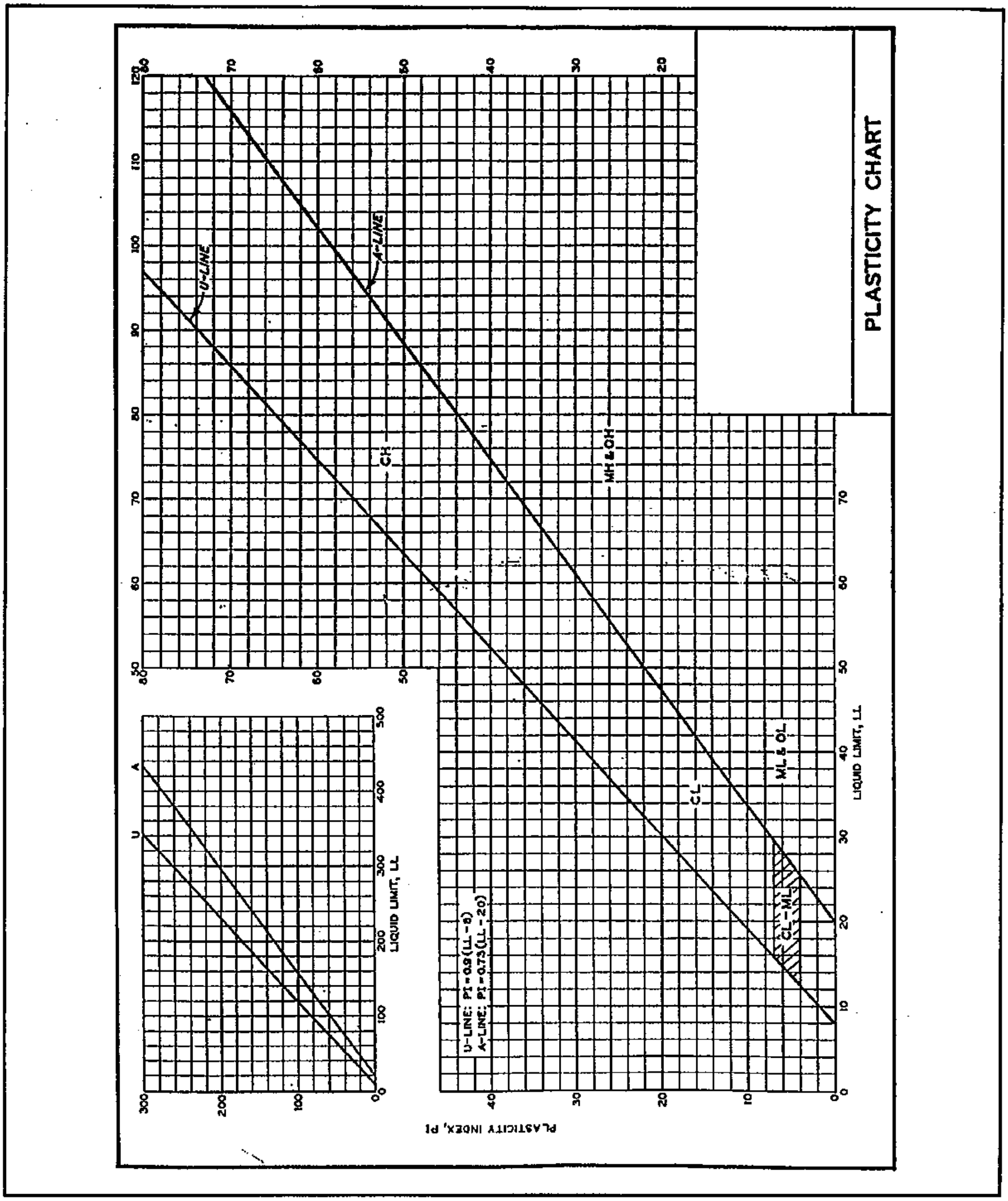
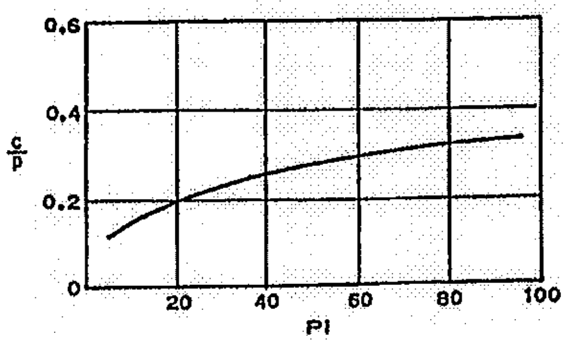
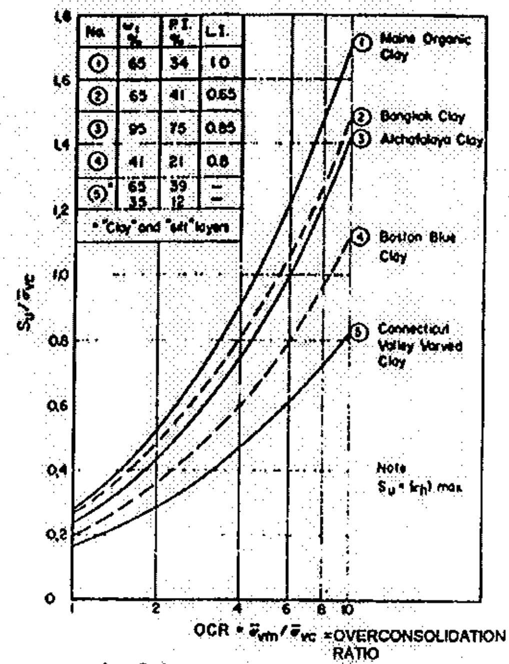


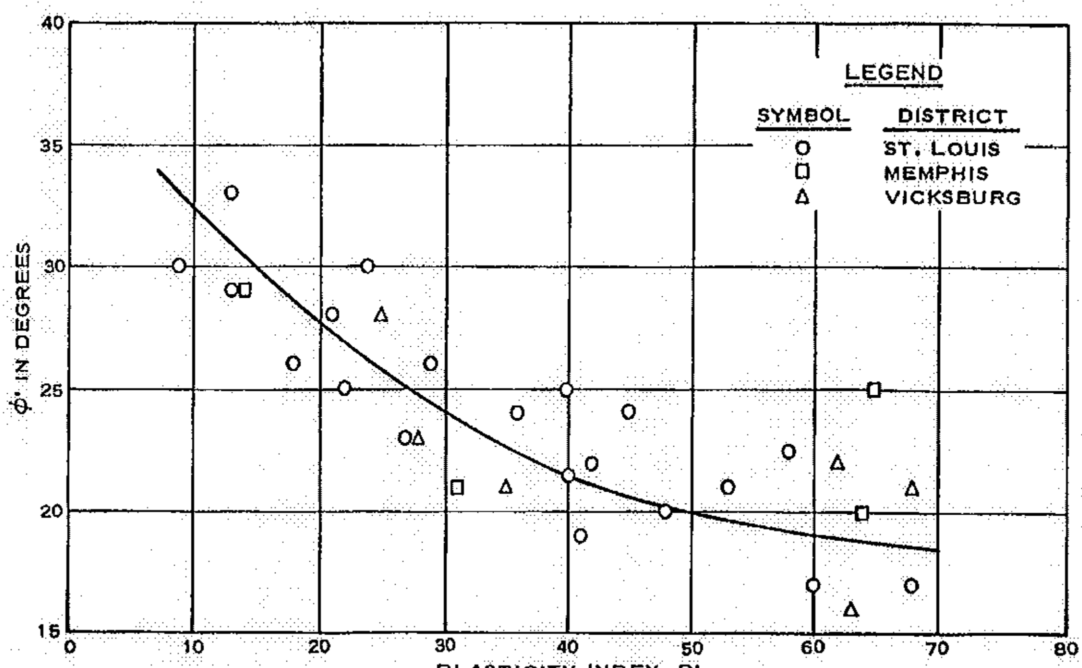
Figure 3-1. Plasticity chart (ENG Form 4334)



a. c/p versus plasticity index for normally consolidated soils (after Bjerrum, ref. A-2)

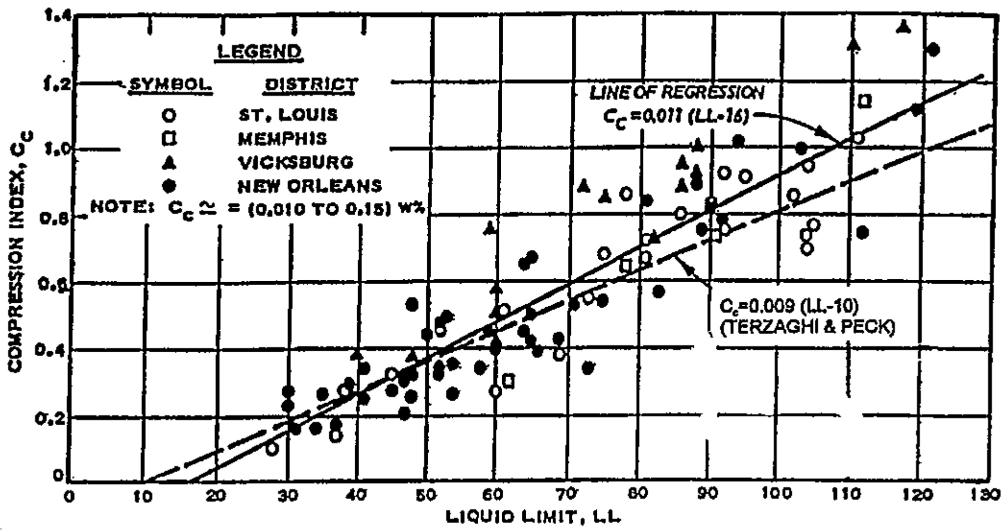


b. S_u/σ_{vc} versus overconsolidation ratio (after Ladd and Foott, 1974, Ref. A-2)

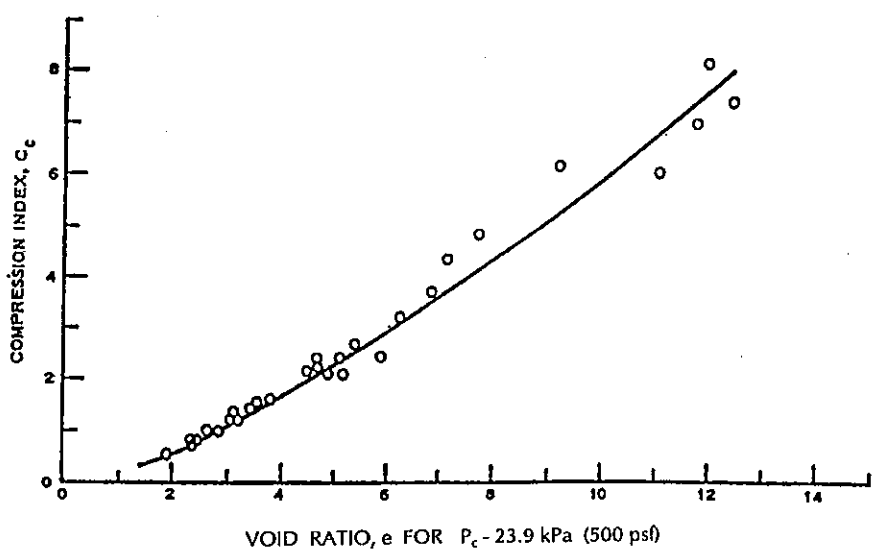


c. ϕ' versus plasticity index

Figure 3-2. Example correlations of strength characteristics for fine-grained soils

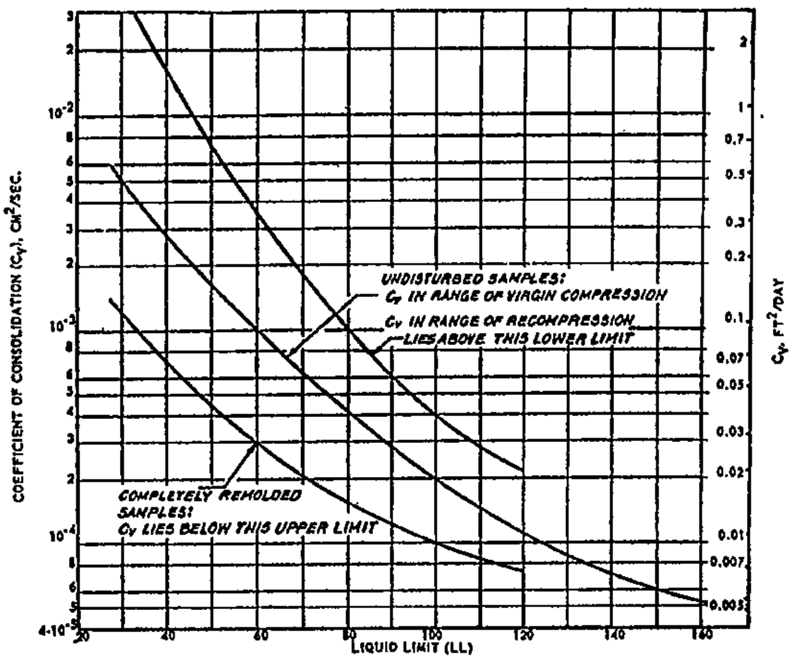


a. Compression index versus liquid limit for normally consolidated soils

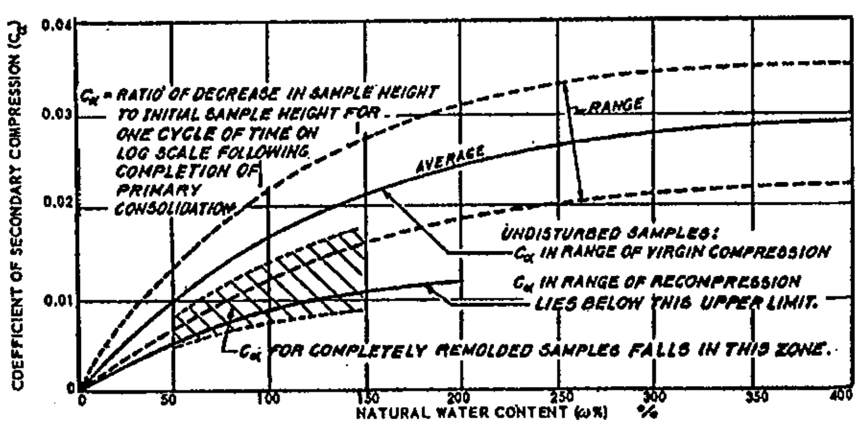


b. Compression index versus initial void ratio for tidal marsh

Figure 3-3. Example correlations for consolidation characteristics of fine-grained soils (after Kapp, ref. A-2)



c. Coefficient of consolidation versus liquid limit (from NAVFAC DM-7 ref. A-1)



d. Coefficient of secondary compression versus water content (from NAVFAC DM-7 ref. A-1)

Figure 3-3. (Concluded)

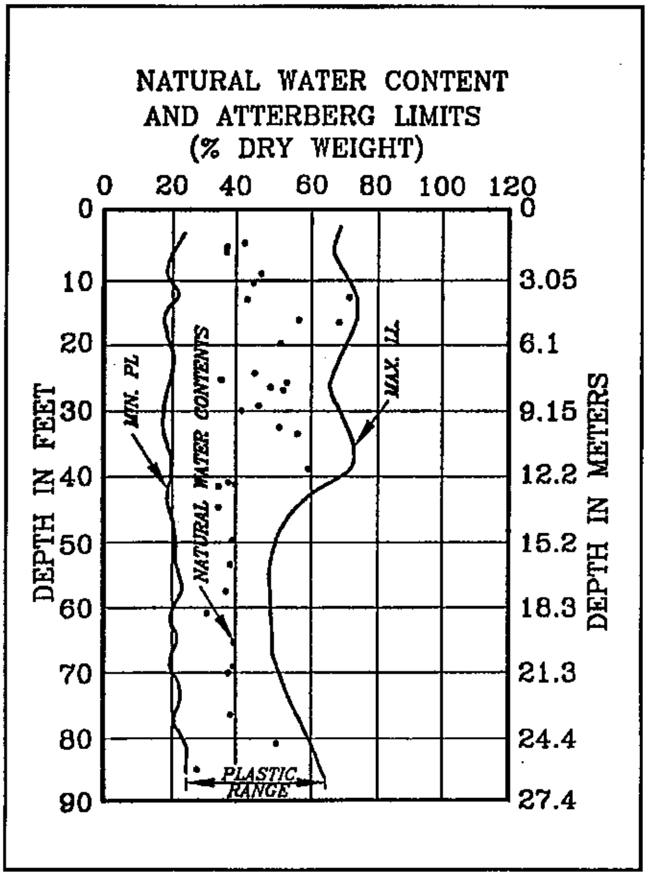


Figure 3-4. Comparisons of Atterberg limits and natural water contents

3-4. Shear Strength

Approximate shear strengths of fine-grained cohesive soils can be rapidly determined on undisturbed foundation samples, and occasionally on reasonably intact samples from disturbed drive sampling, using simple devices such as the pocket penetrometer, laboratory vane shear device, or the miniature vane shear device (Torvane). To establish the reliability of these tests, it is desirable to correlate them with unconfined compression tests. Unconfined compression tests are somewhat simpler to perform than Q triaxial compression tests, but test results exhibit more scatter. Unconfined compression tests are appropriate primarily for testing saturated clays which are not jointed or slickensided. Of the triaxial compression tests, the Q test is the one most commonly performed on foundation clays, since the in situ undrained shear strength generally controls embankment design on such soils. However, where embankments are high, stage construction is being considered, or important structures are located in a levee system, R triaxial compression tests and S direct shear tests should also be performed.

3-5. Consolidation

Consolidation tests are performed for those cases listed in Table 3-1. In some locations correlations of liquid limit and natural water content with coefficient of consolidation, compression index, and coefficient of secondary compression can be used satisfactorily for making estimates of consolidation of foundation clays under load.

3-6. Permeability

Generally there is no need for laboratory permeability tests on fine-grained fill materials, nor on surface clays overlying pervious foundation deposits. In underseepage analyses, simplifying assumptions must be made relative to thickness and soil type of fine-grained surface blankets. Furthermore, animal burrows, root channels, and other discontinuities in surface blankets can significantly affect the overall effective permeability. Therefore, an average value of the coefficient of permeability based on the dominant soil type (Appendix B) is generally of sufficient accuracy for use in underseepage analyses, and laboratory tests are not essential.

3-7. Compaction Tests

The type and number of compaction tests will be influenced by the method of construction and the variability of available borrow materials. The types of compaction tests required are summarized in Table 3-1.

Section II
Coarse-Grained Soils

3-8. Shear Strength

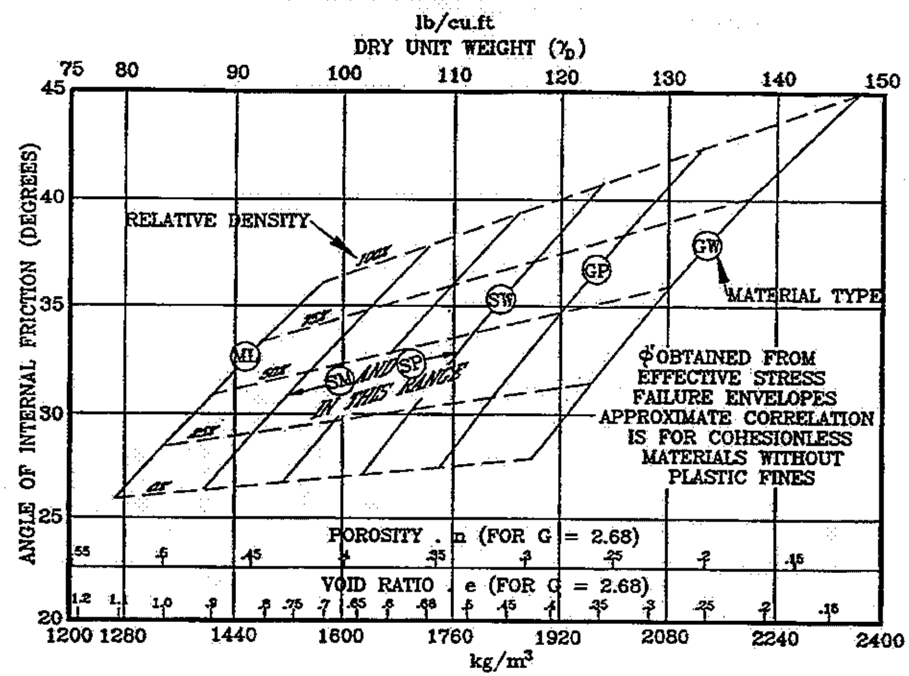
When coarse-grained soils contain few fines, the consolidated drained shear strength is appropriate for use in all types of analyses. In most cases, conservative values of the angle of internal friction (φ) can be assumed from correlations such as those shown in Figure 3-5, and no shear tests will be needed.

3-9. Permeability

To solve the problem of underseepage in levee foundations, reasonable estimates of permeability of pervious foundation deposits are required. However, because of difficulty and expense in obtaining undisturbed samples of sands and gravels, laboratory permeability tests are rarely performed on foundation sands. Instead, field pumping tests or correlations such as that of Figure 3-5 developed between a grain-size parameter (such as D_{10}) and the coefficient of permeability, k , are generally utilized.

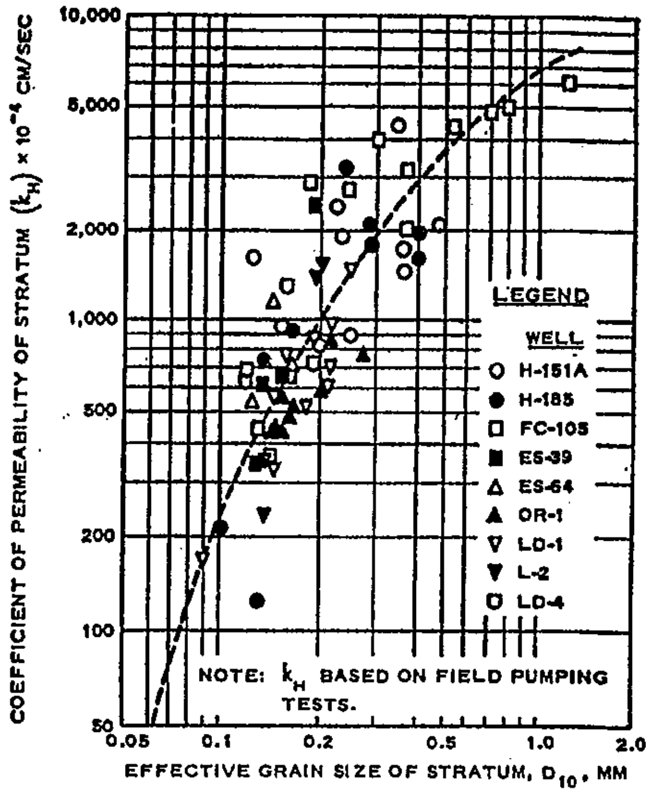
3-10. Density Testing of Pervious Fill

Maximum density tests on available pervious borrow materials should be performed in accordance with ASTM D 4253 so that relative compaction requirements for pervious fills may be checked in the field when required by the specification. Due to the inconsistencies in duplicating minimum densities (ASTM D 4254), relative density may not be used. Factors such as (but not limited to) site specific materials, availability of testing equipment and local practice may make it more practical to utilize methods other than ASTM D 4253 and ASTM D 4254 to control the degree of compaction of cohesionless material. The other methods used include comparison of in-place density to either the maximum Proctor density or the maximum density obtained by ASTM 4253 (if vibratory table is available).



a. Angle of internal friction versus unit weight (from NAVFAC DM-7 ref. A-1)

Figure 3-5. Example correlations for properties of coarse-grained soils



b. Effective grain size, D_{10} versus coefficient of permeability, k_h (from WES TM No. 3-424, ref. A-1)

Figure 3-5. (Concluded)

Chapter 4
Borrow Areas

4-1. General

In the past borrow areas were selected largely on the basis of material types and quantities and haul distances. Today, borrow areas receive much more attention and must be carefully planned and designed, because of considerations such as environmental aspects, increasing land values, and greater recognition of the effects of borrow areas with respect to underseepage, uplift pressures, overall levee stability, and erosion. The following paragraphs discuss some factors involved in locating and using borrow areas.

4-2. Available Borrow Material

a. Material type. Almost any soil is suitable for constructing levees, except very wet, fine-grained soils or highly organic soils. In some cases, though, even these soils may be considered for portions of levees. Accessibility and proximity are often controlling factors in selecting borrow areas, although the availability of better borrow materials involving somewhat longer haul distances may sometimes lead to the rejection of poorer but more readily available borrow.

b. Natural water content. Where compacted levees are planned, it is necessary to obtain borrow material with water content low enough to allow placement and adequate compaction. The cost of drying borrow material to suitable water contents can be very high, in many cases exceeding the cost of longer haul distances to obtain material that can be placed without drying. Borrow soils undergo seasonal water content variations; hence water content data should be based on samples obtained from borrow areas in that season of the year when levee construction is planned. Possible variation of water contents during the construction season should also be considered.

4-3. General Layout

Generally, the most economical borrow scheme is to establish pits parallel and adjacent to the levee. If a levee is adjacent to required channel excavation, levee construction can often utilize material from channel excavation. Large centralized borrow areas are normally established only for the construction of urban levees, where adjacent borrow areas are unavailable. Long, shallow borrow areas along the levee alignment are more suitable, not only because of the shorter haul distance involved, but also because they better satisfy environmental considerations.

a. Location. Where possible, borrow area locations on the river side of a levee are preferable as borrow pits. Borrow area locations within the protected area are less desirable environmentally, as well as generally being more expensive. Riverside borrow locations in some areas will be filled eventually by siltation, thereby obliterating the man-made changes in the landscape. While riverside borrow is generally preferable, required landside borrow from ponding areas, ditches, and other excavations should be used wherever possible. A berm should be left in place between the levee toe and the near edge of the borrow area. The berm width depends primarily on foundation conditions, levee height, and amount of land available. Its width should be established by seepage analyses where pervious foundation material is close to the bottom of the borrow pit and by stability analyses where the excavation slope is near the levee. Minimum berm widths used frequently in the past are 12.2 m (40 ft) riverside and 30.5 m (100 ft) landside, but berm widths should be the maximum practicable since borrow areas may increase the severity of underseepage effects. In borrow area excavation, an adequate thickness of impervious cover should be left over underlying pervious material. For riverside pits a minimum of 0.91 m (3 ft) of cover should be left in place, and for landside pits the cover thickness should be adequate to prevent the formation of boils under expected hydraulic heads. Topsoil from borrow and levee foundation stripping can be stockpiled and spread over the excavated area after borrow excavation has been completed. This reinforces the impervious cover and provides a good base for vegetative growth.

b. Size and shape. It is generally preferable to have riverside borrow areas "wide and shallow" as opposed to "narrow and deep." While this may require extra right-of-way and a longer haul distance, the benefits derived from improved underseepage, hydraulic, and environmental conditions usually outweigh the extra cost. In computing required fill quantities, a shrinkage factor of at least 25 percent should be applied (i.e., borrow area volumes should be at least 125 percent of the levee cross-section volume). This will allow for material shrinkage, and hauling and other losses. Right-of-way requirements should be established about 4.6 to 6.1 m (15 to 20 ft) beyond the top of the planned outer slope of the borrow pit. This extra right-of-way will allow for flattening or caving of the borrow slopes, and can provide maintenance borrow if needed later.

4-4. Design and Utilization

a. Slopes. Excavation slopes of borrow areas should be designed to assure stability. This is particularly important for slopes adjacent to the levee but could also be important for any slope whose top is near the right-of-way limits. Borrow area slopes must also be flat enough to allow mowing, if required. Also, where landside pits are to be placed back into cultivation, changes in grade must be gentle enough to allow farm equipment to operate safely. The slopes of the upstream and downstream ends of riverside pits should be flat enough to avoid erosion when subjected to flow at high water stages.

b. Depths. Depths to which borrow areas are excavated will depend upon factors such as (1) groundwater elevation, (2) changes at depth to undesirable material, (3) preservation of adequate thickness of riverside blanket, and (4) environmental considerations.

c. Foreshore. The foreshore is that area between the riverside edge of the borrow area and the riverbank as shown in Figure 4-1. If a foreshore is specified (i.e., the borrow excavation is not to be cut into the riverbank), it should have a substantial width, say 61 m (200 ft) or more, to help prevent migration of the river channel into the borrow area.

d. Traverse. A traverse is an unexcavated zone left in place at intervals across the borrow area (Figure 4-1). Traverses provide roadways across the borrow area, provide foundations for transmission towers and utility lines, prevent less than bank-full flows from coursing unchecked through the borrow area, and encourage material deposition in the borrow area during high water. Experience has shown that when traverses are overtopped or breached, severe scour damage can result unless proper measures are taken in their design. Traverse heights should be kept as low as possible above the bottom of the pit when they will be used primarily as haul roads. In all cases, flat downstream slopes (on the order of 1V and 6H to 10H) should be specified to minimize scour from overtopping. If the traverse carries a utility line or a public road, even flatter slopes and possibly stone protection should be considered.

e. Drainage. Riverside borrow areas should be so located and excavated that they will fill slowly on a rising river and drain fully on a falling river. This will minimize scour in the pit when overbank river stages occur, promote the growth of vegetation, and encourage silting where reclamation is possible. The bottom of riverside pits should be sloped to drain away from the levee. Culvert pipes should be provided through traverses, and foreshore areas should be ditched through to the river as needed for proper drainage. Landside pits should be sloped to drain away from or parallel to the levee with ditches provided as necessary to outlet

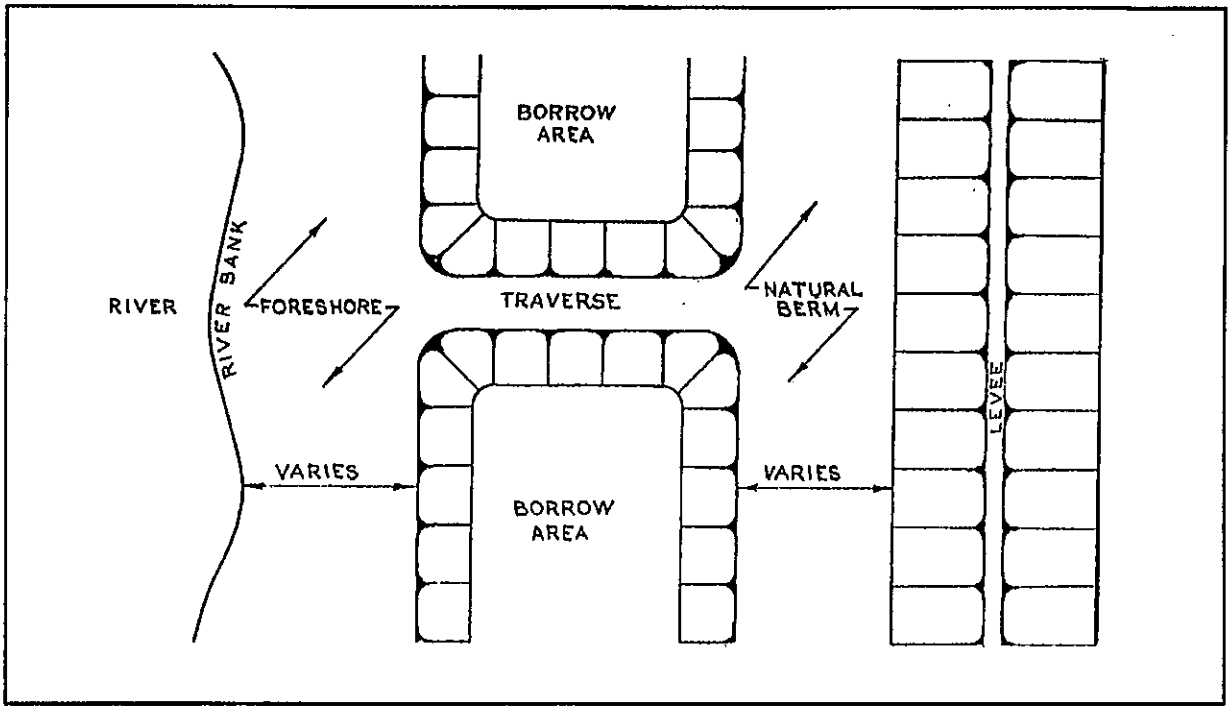


Figure 4-1. Plan of typical levee and borrow areas with traverse and foreshore

points. Gravity outlets or pump stations should be located so as to minimize lengths of flow paths within the pit area.

f. Flow conditions. To avoid damage from confined or restricted flow through the riverside borrow areas, obstructions or impediments to smooth and uniform flow should be removed if possible, or else protective measures must be taken. Riverside borrow areas should be made as uniform in width and grade as possible, avoiding abrupt changes. Removal of obstructions that could cause concentrated flow includes degradation of old levee remnants and of narrow high ground ridges beyond the borrow area, as well as removal of timber from traverses and from foreshore areas immediately adjacent to the borrow area. Obstructions to flow that cannot be removed include transmission towers, bridge piers, and other permanent structures near the levee. In such areas, stone protection should be provided for the levee or borrow area slopes if scour damage is considered probable.

g. Environmental aspects. The treatment of borrow areas after excavation to satisfy aesthetic and environmental considerations has become standard practice. The extent of treatment will vary according to the type and location of a project. Generally, projects near urban areas or where recreational areas are to be developed will require more elaborate treatment than those in sparsely populated agricultural areas. Minimum treatment should include proper drainage, topographic smoothing, and the promotion of conditions conducive to vegetative growth. Insofar as practicable, borrow areas should be planted to conform to the surrounding landscape. Stands of trees should be left remaining on landside borrow areas if at all possible, and excavation procedures should not leave holes, trenches, or abrupt slopes. Restoration of vegetative growth is important for both landside and riverside pits as it is not only pleasing aesthetically but serves as protection against erosion. Willow trees can aid considerably in drying out boggy areas. Riverside pits should not be excavated so deep that restored grass cover will be drowned out by long submergence.

Agencies responsible for maintenance of completed levees should be encouraged to plant and maintain vegetation, including timber, in the borrow areas. It is desirable that riverside borrow pits be filled in by natural processes, and frequent cultivation of these areas should be discouraged or prohibited, if possible, until this has been achieved. Guidelines for landscape planting are given in EM 1110-2-301.

h. Clearing, grubbing, and stripping. Borrow areas should be cleared and grubbed to the extent needed to obtain fill material free of objectionable matter, such as trees, brush, vegetation, stumps, and roots. Subareas within borrow areas may be specified to remain untouched to preserve standing trees and existing vegetation. Topsoil with low vegetative cover may be stripped and stockpiled for later placement on outer landside slopes of levees and seepage berms.

Chapter 5
Seepage Control

Section I
Foundation Underseepage

5-1. General

Without control, underseepage in pervious foundations beneath levees may result in (a) excessive hydrostatic pressures beneath an impervious top stratum on the landside, (b) sand boils, and (c) piping beneath the levee itself. Underseepage problems are most acute where a pervious substratum underlies a levee and extends both landward and riverward of the levee and where a relatively thin top stratum exists on the landside of the levee. Principal seepage control measures for foundation underseepage are (a) cutoff trenches, (b) riverside impervious blankets, (c) landside seepage berms, (d) pervious toe trenches, and (e) pressure relief wells. These methods will be discussed generally in the following paragraphs. Detailed design guidance is given in Appendixes B and C. Turnbull and Mansur (1959) have proposed control measures for underseepage also. Additional information on seepage control in earth foundations including cutoffs, impervious blankets, seepage berms, relief wells and trench drains is given in EM 1110-2-1901 and EM 1110-2-1914.

5-2. Cutoffs

A cutoff beneath a levee to block seepage through pervious foundation strata is the most positive means of eliminating seepage problems. Positive cutoffs may consist of excavated trenches backfilled with compacted earth or slurry trenches usually located near the riverside toe. Since a cutoff must penetrate approximately 95 percent or more of the thickness of pervious strata to be effective, it is not economically feasible to construct cutoffs where pervious strata are of considerable thickness. For this reason cutoffs will rarely be economical where they must penetrate more than 12.2 m (40 ft). Steel sheet piling is not entirely watertight due to leakage at the interlocks but can significantly reduce the possibility of piping of sand strata in the foundation. Open trench excavations can be readily made above the water table, but if they must be made below the water table, well point systems will be required. Cutoffs made by the slurry trench method (reference Appendix A) can be made without a dewatering system, and the cost of this type of cutoff should be favorable in many cases in comparison with costs of compacted earth cutoffs.

5-3. Riverside Blankets

Levees are frequently situated on foundations having natural covers of relatively fine-grained impervious to semipervious soils overlying pervious sands and gravels. These surface strata constitute impervious or semipervious blankets when considered in connection with seepage control. If these blankets are continuous and extend riverward for a considerable distance, they can effectively reduce seepage flow and seepage pressures landside of the levee. Where underseepage is a problem, riverside borrow operations should be limited in depth to prevent breaching the impervious blanket. If there are limited areas where the blanket becomes thin or pinches out entirely, the blanket can be made effective by placing impervious materials in these areas. The effectiveness of the blanket depends on its thickness, length, distance to the levee riverside toe, and permeability and can be evaluated by flow-net or approximate mathematical solutions, as shown in Appendix B. Protection of the riverside blanket against erosion is important.

5-4. Landside Seepage Berms

a. General. If uplift pressures in pervious deposits underlying an impervious top stratum landward of a levee become greater than the effective weight of the top stratum, heaving and rupturing of the top stratum may occur, resulting in sand boils. The construction of landside berms (where space is available) can eliminate this hazard by providing (a) the additional weight needed to counteract these upward seepage forces and (b) the additional length required to reduce uplift pressures at the toe of the berm to tolerable values. Seepage berms may reinforce an existing impervious or semipervious top stratum, or, if none exists, be placed directly on pervious deposits. A berm also affords some protection against sloughing of the landside levee slope. Berms are relatively simple to construct and require very little maintenance. They frequently improve and reclaim land as areas requiring underseepage treatment are often low and wet. Berms can also serve as a source of borrow for emergency repairs to the levee. Because they require additional fill material and space, they are used primarily with agricultural levees. Subsurface profiles must be carefully studied in selecting berm widths. For example, where a levee is founded on a thin top stratum and thicker clay deposits lie a short distance landward, as shown in Figure 5-1, the berm should extend far enough landward to lap the thick clay deposit, regardless of the computed required length. Otherwise, a concentration of seepage and high exit gradients may occur between the berm toe and the landward edge of the thick clay deposit.

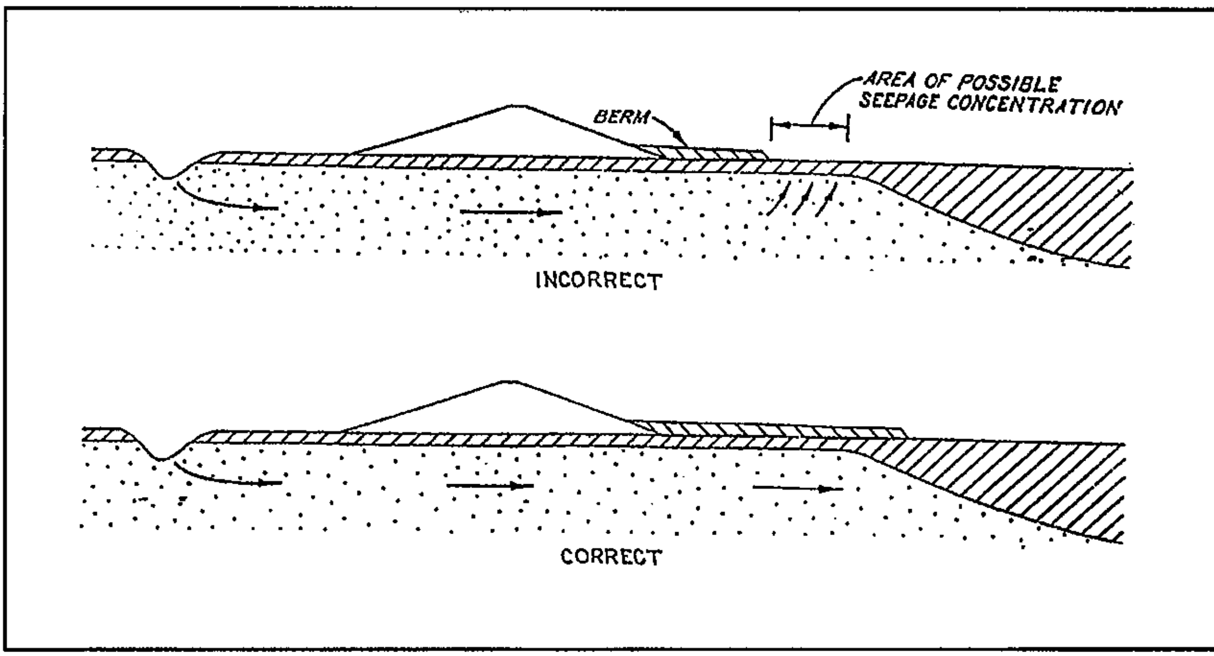


Figure 5-1. Example of incorrect and correct berm length according to existing foundation conditions

b. Types of seepage berms. Four types of seepage berms have been used, with selection based on available fill materials, space available landside of the levee proper, and relative costs.

(1) Impervious berms. A berm constructed of impervious soils restricts the pressure relief that would otherwise occur from seepage flow through the top stratum, and consequently increases uplift pressures

beneath the top stratum. However, the berm can be constructed to the thickness necessary to provide an adequate factor of safety against uplift.

(2) Semipervious berms. Semipervious material used in constructing this type of berm should have an in-place permeability equal to or greater than that of the top stratum. In this type of berm, some seepage will pass through the berm and emerge on its surface. However, since the presence of this berm creates additional resistance to flow, subsurface pressures at the levee toe will be increased.

(3) Sand berms. While a sand berm will offer less resistance to flow than a semipervious berm, it may also cause an increase in substratum pressures at the levee toe if it does not have the capacity to conduct seepage flow landward without excessive internal head losses. Material used in a sand berm should be as pervious as possible, with a minimum permeability of 100×10^{-4} cm per sec. Sand berms require less material and occupy less space than impervious or semipervious berms providing the same degree of protection.

(4) Free-draining berms. A free-draining berm is one composed of random fill overlying horizontal sand and gravel drainage layers (with a terminal perforated collector pipe system), designed by the same methods used for drainage layers in dams. Although the free-draining berm can afford protection against underseepage pressures with less length and thickness than the other types of seepage berms, its cost is generally much greater than the other types, and thus it is rarely specified.

c. Berm design. Design equations, criteria, and examples are presented in Appendix C for seepage berms.

d. Computer programs to use for seepage analysis.

(1) If the soil can be idealized with a top blanket of uniform thickness and seepage flow is assumed to be horizontal in the foundation and vertical in the blanket, then LEVSEEP (Brizendine, Taylor, and Gabr 1995) or LEVEEMSU (Wolff 1989; Gabr, Taylor, Brizendine, and Wolff 1995) could be used.

(2) If the soil profile is characterized by a top blanket and two foundation layers of uniform thickness, and seepage flow is assumed to be horizontal in the foundation, horizontal and vertical in the transition layer, and vertical in the blanket, then LEVEEMSU or the finite element method (CSEEP) could be used (Biedenharn and Tracy 1987; Knowles 1992; Tracy 1994; Gabr, Brizendine, and Taylor 1995). LEVEESMU would be simpler to use.

(3) If the idealized soil profile includes irregular geometry (slopes greater than 1 vertical to 100 horizontal), more than three layers and/or anisotropic permeability ($k_v \neq k_h$), then only the finite element method (CSEEP) is applicable. When using CSEEP it is recommended that FastSEEP, a graphical pre- and post-processor, be used for mesh generation, assigning boundary conditions and soil properties, and viewing the results (Engineering Computer Graphics Laboratory 1996).

5-5. Pervious Toe Trench

a. General. Where a levee is situated on deposits of pervious material overlain by little or no impervious material, a partially penetrating toe trench, as shown in Figure 5-2, can improve seepage conditions at or near the levee toe. Where the pervious stratum is thick, a drainage trench of any practicable depth would attract only a small portion of the seepage flow and detrimental underseepage would bypass the trench. Consequently, the main use of a pervious toe trench is to control shallow underseepage and protect the area in the vicinity of the levee toe. Pervious toe trenches may be used in conjunction with relief well systems;

the wells collect the deeper seepage and the trench collects the shallow seepage. Such a system is shown in Figure 5-3. The trench is frequently provided with a perforated pipe to collect the seepage. The use of a collector system is dependent on the volume of seepage and, to some degree, the general location of the levee. Collector systems are usually not required for agricultural levees but find wider use in connection with urban levees.

b. Location. As seen in Figures 5-2 and 5-3, pervious drainage trenches are generally located at the levee toe, but are sometimes constructed beneath the downstream levee slope as shown in Figure 5-4. Here the trench is located at the landward quarter point of the levee, and discharge is provided through a horizontal pervious drainage layer. Unless it is deep enough, it may allow excessive seepage pressures to act at the toe. There is some advantage to a location under the levee if the trench serves also as an inspection trench and because the horizontal pervious drainage layer can help to control embankment seepage.

c. Geometry. Trench geometry will depend on the volume of expected underseepage, desired reduction in uplift pressure, construction practicalities, and the stability of the material in which it is being excavated. Trench widths varying from 0.61 to 1.83 m (2 to 6 ft) have been used. Trench excavation can be expedited if a ditching machine can be used. However, narrow trench widths will require special compaction equipment. One such piece of equipment (Figure 5-5), which is a vibrating-plate type of compactor specially made to fit on the boom of a backhoe, has apparently performed satisfactorily.

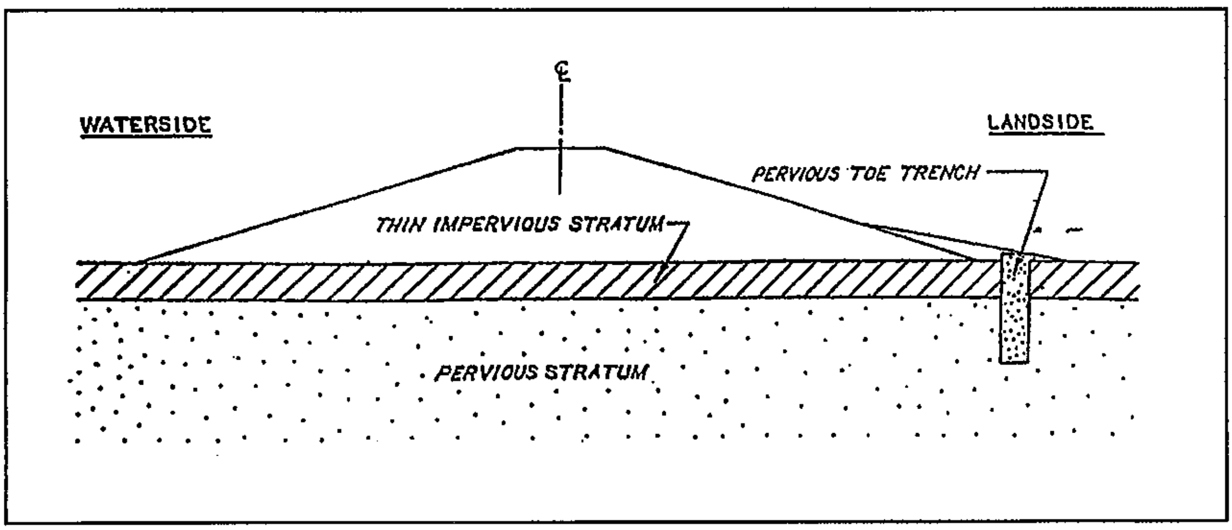


Figure 5-2. Typical partially penetrating pervious toe trench

d. Backfill. The sand backfill for trenches must be designed as a filter material in accordance with criteria given in Appendix D. If a collector pipe is used, the pipe should be surrounded by about a 305-mm (1-ft) thickness of gravel having a gradation designed to provide a stable transition between the sand backfill and the perforations or slots in the pipe. A typical section of a pervious drainage trench with collector pipe is shown in Figure 5-6. Placement of trench backfill must be done in such a manner as to minimize segregation. Compaction of the backfill should be limited to prevent breakdown of material or over compaction resulting in lowered permeabilities.

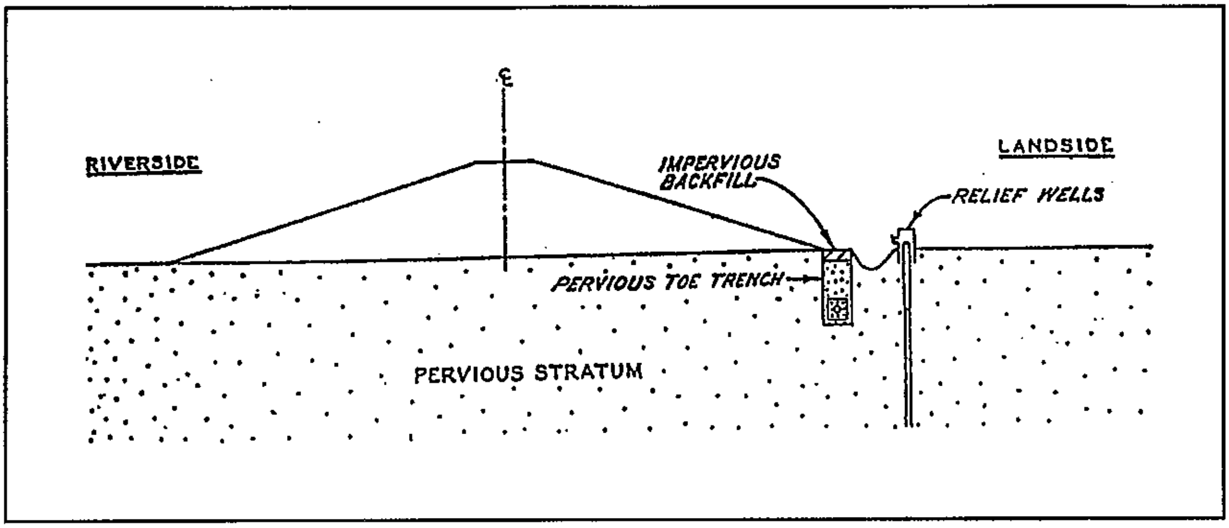


Figure 5-3. Typical pervious toe trench with collector pipe (Figure 5-6 shows trench details)

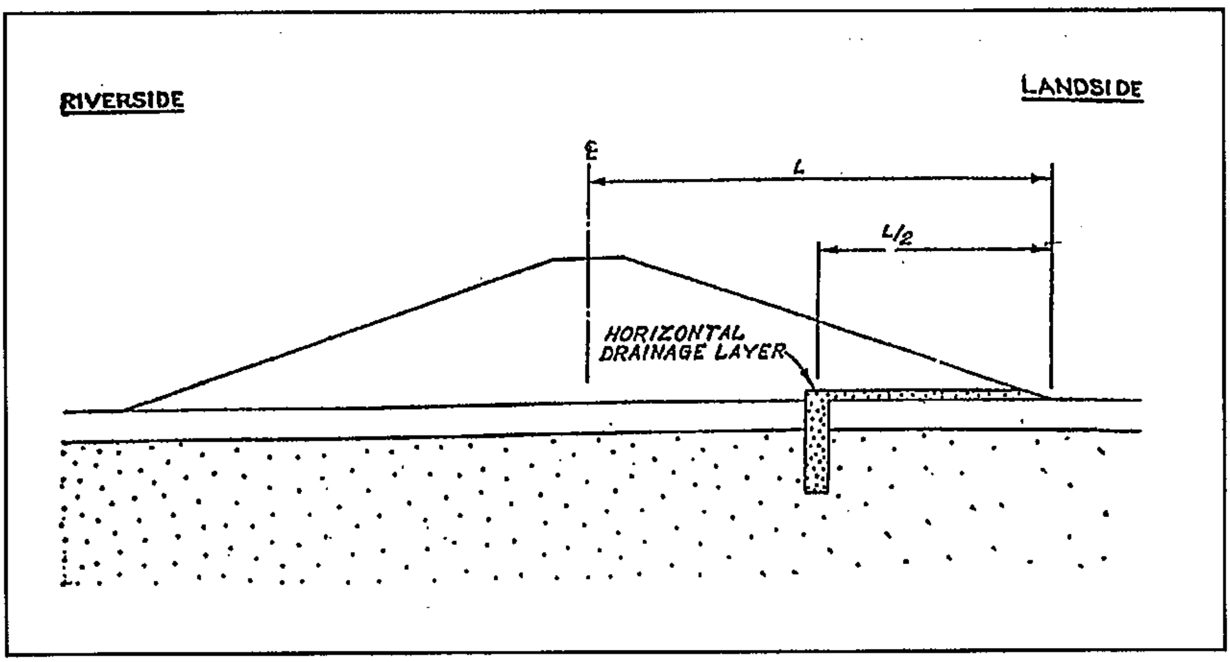


Figure 5-4. Pervious toe trench located beneath landward slope

5-6. Pressure Relief Wells

a. General. Pressure relief wells may be installed along the landside toe of levees to reduce uplift pressure which may otherwise cause sand boils and piping of foundation materials. Wells accomplish this by intercepting and providing controlled outlets for seepage that would otherwise emerge uncontrolled landward of the levee. Pressure relief well systems are used where pervious strata underlying a levee are

Figure 5-5. Special equipment for compacting sand in pervious toe trenches

too deep or too thick to be penetrated by cutoffs or toe drains or where space for landside berms is limited. Relief wells should adequately penetrate pervious strata and be spaced sufficiently close to intercept enough seepage to reduce to safe values the hydrostatic pressures acting beyond and between the wells. The wells must offer little resistance to the discharge of water while at the same time prevent loss of any soil. They must also be capable of resisting corrosion and bacterial clogging. Relief well systems can be easily expanded if the initial installation does not provide the control needed. Also, the discharge of existing wells can be increased by pumping if the need arises. A relief well system requires a minimum of additional real estate as compared with the other seepage control measures such as berms. However, wells require periodic maintenance and frequently suffer loss in efficiency with time, probably due to clogging of well screens by muddy surface waters, bacteria growth, or carbonate incrustation. They increase seepage discharge, and means for collecting and disposing of their discharge must be provided.

b. Design of well systems. The design of a pressure relief well system involves determination of well spacing, size, and penetration to reduce uplift between wells to allowable values. Factors to be considered are (a) depth, stratification, and permeability of foundation soils, (b) distance to the effective source of seepage, (c) characteristics of the landside top stratum, if any, and (d) degree of pressure relief desired. Guidance on the method used to determine well spacing, size, and penetration is contained in EM 1110-2-1914 and U.S. Army Engineer Waterways Experiment Station TM No. 3-424. Where no control measures are present, relief wells for agricultural and urban levees should be designed so that i_{max} midway between the wells or landward from the well line should not exceed 0.50 (equivalent to FS = 1.7 for an average soil saturated unit weight of 1840 kg/m^3 (115 pcf)). Many combinations of well spacing and penetration will produce the desired pressure relief; hence, the final selected spacing and penetration must be based on cost comparisons of alternative combinations. After the general well spacing for a given reach of levee has been determined, the actual location of each well should be established to ensure that the wells will be located at critical seepage points and will fit natural topographic features.

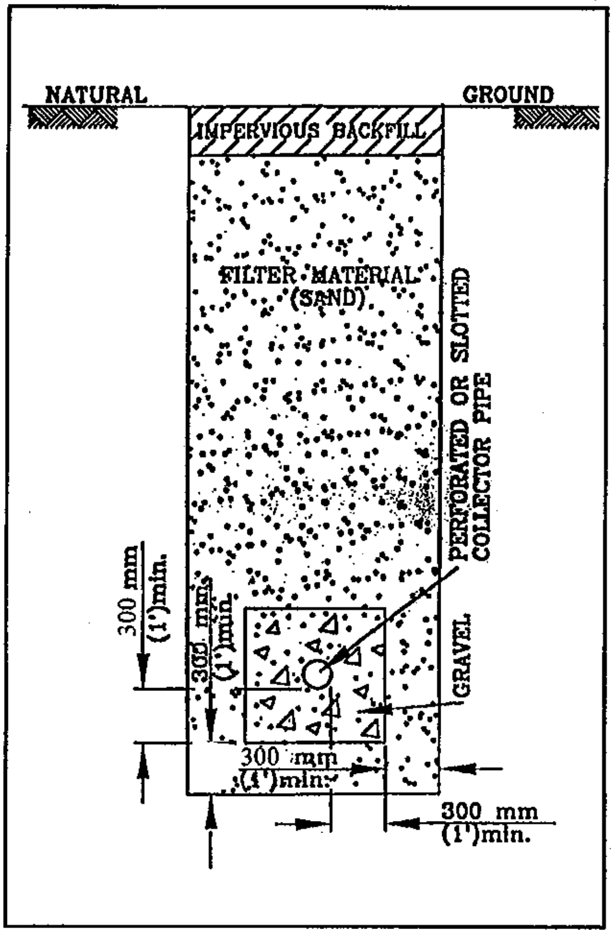


Figure 5-6. Pervious toe trench with collector pipe

c. Design of individual wells. The design of the well involves the selection of type and length of riser pipe and screen, design of the gravel pack, and design of well appurtenances. A widely used well design that has given good service in the past is shown in Figure 5-7.

(1) Riser pipe and screen. The well screen normally extends from just below the top of the pervious stratum to the bottom of the well, with solid riser pipe installed from the top of the pervious strata to the surface. In zones of very fine sand or silt, the screen is replaced by unperforated (blank) pipe. The type of material for the riser and screen should be selected only after a careful study of the corrosive properties of the water to be carried by the well. Many types of metals, alloys, fiberglass, plastics, and wood have been used in the past. At the present time, stainless steel and plastic are the most widely used, primarily because of their corrosion-resistant properties. Plastic risers should be considered with caution, being susceptible to damages during mechanical treatment or chemical treatment which develop excessive heat or cold.

(2) Filter. The filter that surrounds the screen must be designed in accordance with criteria given in Appendix D using the slot size of the screen and the gradation of surrounding pervious deposit as a basis of design. No matter what size screen is used, a minimum of 152.4 mm (6 in.) of filter material should surround the screen and the filter should extend a minimum of 610.8 mm (2 ft) above the top and 1.2 m (4 ft) below the bottom of the well screen. Above the filter to the bottom of the concrete or impervious backfill, sand backfill may be used.

(3) Well appurtenances. In selecting well appurtenances, consideration must be given to ease of maintenance, protection against contamination from back flooding, damage by debris, and vandalism. To prevent wells from becoming backflooded with muddy surface water, which greatly impairs their efficiency when they are not flowing, an aluminum check valve, rubber gasket, and plastic standpipe, as shown in Figure 5-7, can be installed on each well. To safeguard against vandalism, accidental damage, and the entrance of debris, the tops of the wells should be provided with a metal screen or flap-type gate. The elevation of the top of any protective standpipes must be used in design as the well discharge elevation.

d. Well installation. Proper methods of drilling, backfilling, and developing a relief well must be employed or the well will be of little or no use. These procedures are described in detail in EM 1110-2-1914.

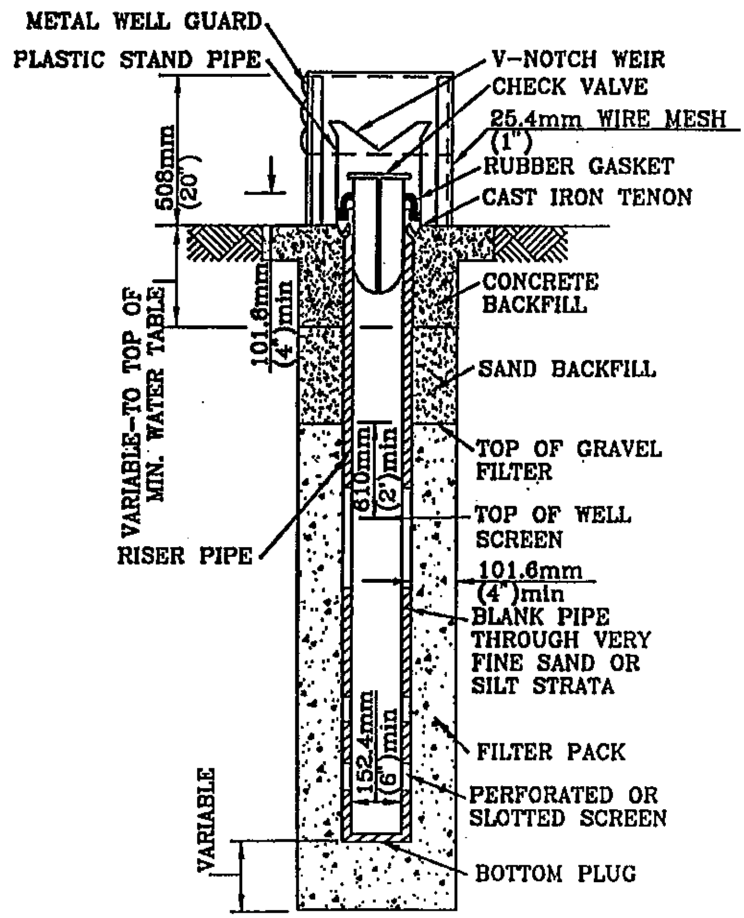
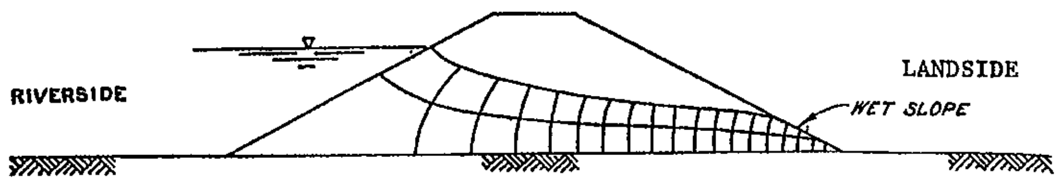


Figure 5-7. Typical relief well

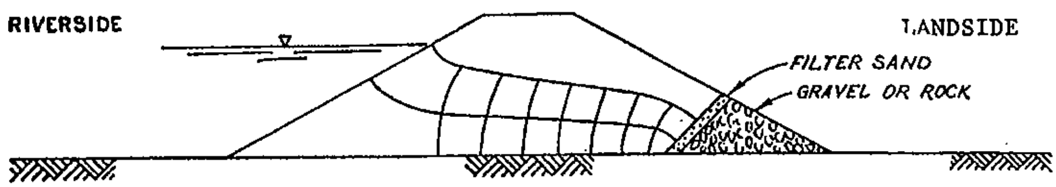
Section II
Seepage Through Embankments

5-7. General

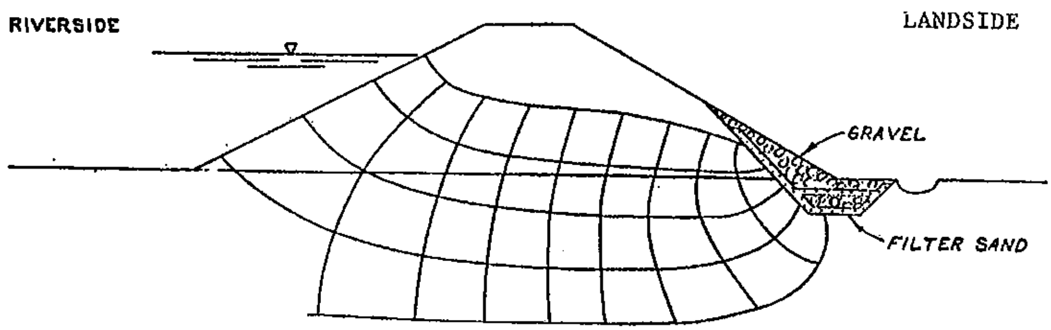
Should through seepage in an embankment emerge on the landside slope (Figure 5-8a), it can soften fine-grained fill in the vicinity of the landside toe, cause sloughing of the slope, or even lead to piping (internal erosion) of fine sand or silt materials. Seepage exiting on the landside slope would also result in high seepage forces, decreasing the stability of the slope. In many cases, high water stages do not act against the levee long enough for this to happen, but the possibility of a combination of high water and a period of heavy precipitation may bring this about. If landside stability berms or berms to control underseepage are required because of foundation conditions, they may be all that is necessary to prevent seepage emergence on the



a. Homogeneous section on impervious foundation seepage emerging on landside slope



b. Section with pervious toe



c. Pervious toe combined with partially penetrating toe trencn

Figure 5-8. Embankment with through seepage

slope. On the other hand, if no berms are needed, landside slopes are steep, and floodstage durations and other pertinent considerations indicate a potential problem of seepage emergence on the slope, provisions should be incorporated in the levee section such as horizontal and/or inclined drainage layers or toe drains to prevent seepage from emerging on the landside slope. These require select pervious granular material and graded filter layers to ensure continued functioning, and therefore add an appreciable cost to the levee construction, unless suitable materials are available in the borrow areas with only minimal processing required. Where large quantities of pervious materials are available in the borrow areas, it may be more practicable to design a zoned embankment with a large landside pervious zone. This would provide an efficient means of through seepage control and good utilization of available materials. Additional information on seepage control in earth embankments including zoning embankments and vertical (or inclined) and horizontal drains is given in Chapter 8 of EM 1110-2-1901.

5-8. Pervious Toe Drain

A pervious toe (Figure 5-8b) will provide a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious toe can also be combined with partially penetrating toe trenches, which have previously been discussed, as a method for controlling shallow underseepage. Such a configuration is shown in Figure 5-8c.

5-9. Horizontal Drainage Layers

Horizontal drainage layers, as shown in Figure 5-9a, essentially serve the same purpose as a pervious toe but are advantageous in that they can extend further under the embankment requiring a relatively small amount of additional material. They can also serve to protect the base of the embankment against high uplift pressures where shallow foundation underseepage is occurring. Sometimes horizontal drainage layers serve also to carry off seepage from shallow foundation drainage trenches some distance under the embankment as shown previously in Figure 5-4.

5-10. Inclined Drainage Layers

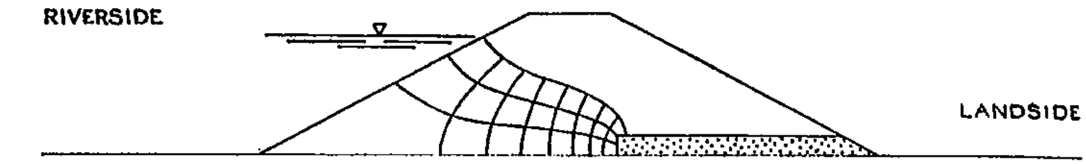
An inclined drainage layer as shown in Figure 5-9b is one of the more positive means of controlling internal seepage and is used extensively in earth dams. It is rarely used in levee construction because of the added cost, but might be justified for short levee reaches in important locations where landside slopes must be steep and other control measures are not considered adequate and the levee will have high water against it for prolonged periods. The effect of an inclined drainage layer is to completely intercept embankment seepage regardless of the degree of stratification in the embankment or the material type riverward or landward of the drain. As a matter of fact, the use of this type of drain allows the landside portion of a levee to be built of any material of adequate strength regardless of permeability. When used between an impervious core and outer pervious shell (Figure 5-9c), it also serves as a filter to prevent migration of impervious fines into the outer shell. If the difference in gradation between the impervious and pervious material is great, the drain may have to be designed as a graded filter (Appendix D). Inclined drains must be tied into horizontal drainage layers to provide an exit for the collected seepage as shown in Figures 5-9b and 5-9c.

5-11. Design of Drainage Layers

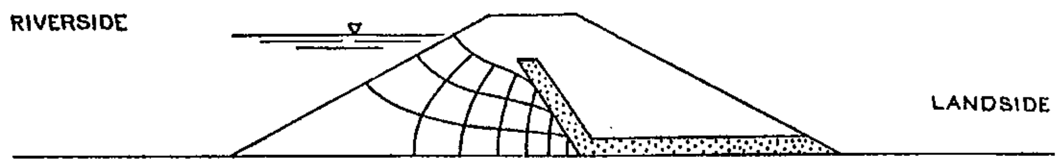
The design of pervious toe drains and horizontal and inclined drainage layers must ensure that such drains have adequate thickness and permeability to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles. The design of drainage layers must satisfy the criteria outlined in Appendix D for filter design. Horizontal drainage layers should have a minimum thickness of 457.2 mm (18 in.) for construction purposes.

5-12. Compaction of Drainage Layers

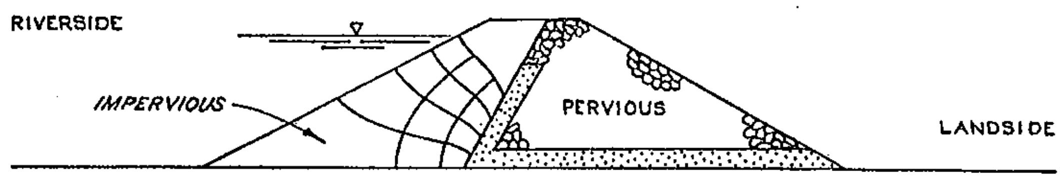
Placement and compaction of drainage layers must ensure that adequate density is attained, but should not allow segregation and contamination to occur. Vibratory rollers are probably the best type of equipment for compaction of cohesionless material although crawler tractors and rubber-tired rollers have also been used successfully. Saturation or flooding of the material as the roller passes over it will aid in the compaction process and in some cases has been the only way specified densities could be attained. Care must always be taken to not overcompact to prevent breakdown of materials or lowering of expected permeabilities. Loading, dumping, and spreading operations should be observed to ensure that segregation does not occur. Gradation tests should be run both before and after compaction to ensure that the material meets specifications and does not contain too many fines.



a. **Horizontal drainage layer**



b. **Inclined drainage layer-homogeneous embankment**



c. **Inclined drainage layer-zoned embankment**

Figure 5-9. Use of horizontal and inclined drainage layers to control seepage through an embankment

Chapter 6
Slope Design and Settlement

Section I
Embankment Stability

6-1. Embankment Geometry

a. Slopes. For levees of significant height or when there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis. Low levees and levees to be built of good material resting on proven foundations may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of levee slopes.

(1) Type of construction. Fully compacted levees generally enable the use of steeper slopes than those of levees constructed by semicompacted or hydraulic means. In fact, space limitations in urban areas often dictate minimum levee sections requiring select material and proper compaction to obtain a stable section.

(2) Ease of construction. A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers.

(3) Maintenance. A 1V on 3H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment and walked on during inspections.

(4) Seepage. For sand levees, a 1V on 5H landside slope is considered flat enough to prevent damage from seepage exiting on the landside slope.

(5) Slope protection. Riverside slopes flatter than those required for stability may have to be specified to provide protection from damage by wave action.

b. Final Levee Grade. In the past, freeboard was used to account for hydraulic, geotechnical, construction, operation and maintenance uncertainties. The term and concept of freeboard to account for these uncertainties is no longer used in the design of levee projects. The risk-based analysis directly accounts for hydraulic uncertainties and establishes a nominal top of protection. Deterministic analysis using physical properties of the foundation and embankment materials should be used to set the final levee grade to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances.

c. Crown width. The width of the levee crown depends primarily on roadway requirements and future emergency needs. To provide access for normal maintenance operations and floodfighting operations, minimum widths of 3.05 to 3.66 m (10 to 12 ft) are commonly used with wider turnaround areas provided at specified intervals; these widths are about the minimum feasible for construction using modern heavy earthmoving equipment and should always be used for safety concerns. Where the levee crown is to be used as a higher class road, its width is usually established by the responsible agency.

6-2. Standard Levee Sections and Minimum Levee Section

a. Many districts have established standard levee-sections for particular levee systems, which have proven satisfactory over the years for the general stream regime, foundation conditions prevailing in those areas, and for soils available for levee construction. For a given levee system, several different standard sections may be established depending on the type of construction to be used (compacted, semicompacted, uncompacted, or hydraulic fill). The use of standard sections is generally limited to levees of moderate height (say less than 7.62 m (25 ft)) in reaches where there are no serious underseepage problems, weak foundation soils, or undesirable borrow materials (very wet or very organic). In many cases the standard levee section has more than the minimum allowable factor of safety relative to slope stability, its slopes being established primarily on the basis of construction and maintenance considerations. Where high levees or levees on foundations presenting special underseepage or stability problems are to be built, the uppermost riverside and landside slopes of the levee are often the same as those of the standard section, with the lower slopes flattened or stability berms provided as needed.

b. The adoption of standard levee sections does not imply that stability and underseepage analyses are not made. However, when borings for a new levee clearly demonstrate foundation and borrow conditions similar to those at existing levees, such analyses may be very simple and made only to the extent necessary to demonstrate unquestioned levee stability. In addition to being used in levee design, the standard levee sections are applicable to initial cost estimate, emergency and maintenance repairs.

c. The minimum levee section shall have a crown width of at least 3.05 m (10 ft) and a side slope flatter than or equal to 1V on 2H, regardless of the levee height or the possibly less requirements indicated in the results of stability and seepage analyses. The required dimensions of the minimum levee section is to provide an access road for flood-fighting, maintenance, inspection and for general safety conditions.

6-3. Effects of Fill Characteristics and Compaction

a. Compacted fills. The types of compaction, water content control, and fill materials govern the steepness of levee slopes from the stability aspect if foundations have adequate strength. Where foundations are weak and compressible, high quality fill construction is not justified, since these foundations can support only levees with flat slopes. In such cases uncompacted or semicompacted fill, as defined in paragraph 1-5, is appropriate. Semicompacted fill is also used where fine-grained borrow soils are considerably wet of optimum or in construction of very low levees where other considerations dictate flatter levee slopes than needed for stability. Uncompacted fill is generally used where the only available borrow is very wet and frequently has high organic content and where rainfall is very high during the construction season. When foundations have adequate strength and where space is limited in urban areas both with respect to quantity of borrow and levee geometry, compacted levee fill construction by earth dam procedures is frequently selected. This involves the use of select material, water content control, and compaction procedures as described in paragraph 1-5.

b. Hydraulic Fill. Hydraulic fill consists mostly of pervious sands built with one or two end-discharge or bottom-discharging pipes. Tracked or rubber-tired dozers or front-end loaders are used to move the sand to shape the embankment slopes. Because a levee constructed of hydraulic fill would be very pervious and have a low density, it would require a large levee footprint and would be susceptible to soil liquefaction. Hydraulic fill would also quickly erode upon overtopping or where an impervious covering was penetrated. For these reasons, hydraulic fill may be used for stability berms, pit fills and seepage berms but shall not normally be used in constructing levee embankments. However, hydraulic fill may be used for levees protecting agricultural areas whose failure would not endanger human life and for zoned embankments that include impervious seepage barriers.

Section II
Stability Analyses

6-4. Methods of Analysis

The principal methods used to analyze levee embankments for stability against shear failure assume either (a) a sliding surface having the shape of a circular arc within the foundation and/or the embankment or (b) a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface. Various methods of analysis are described in EM 1110-2-1902, and can be chosen for use where determined appropriate by the designer. Computer programs are available for these analyses, with the various loading cases described in EM 1110-2-1902, so the effort of making such analyses is greatly reduced, and primary attention can be devoted to the more important problems of defining the shear strengths, unit weights, geometry, and limits of possible sliding surfaces.

6-5. Conditions Requiring Analysis

The various loading conditions to which a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; Case III, steady seepage from full flood stage, fully developed phreatic surface; Case IV, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given. For more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case refer to EM 1110-2-1902.

a. Case I - End of construction. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

b. Case II - Sudden drawdown. This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. For the selection of the shear strengths see Table 6-1a.

c. Case III - Steady seepage from full flood stage (fully developed phreatic surface). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability. Design shear strengths should be based on Table 6-1a.

d. Case IV - Earthquake. Earthquake loadings are not normally considered in analyzing the stability of levees because of the low probability of earthquake coinciding with periods of high water. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required.

Table 6-1a
Summary of Design Conditions

Analysis Condition	Shear Strength[a]	Pore Water Pressure
During and End-of-Construction	Free draining soils - use effective stresses	Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
	Low permeability soils - use undrained strengths and total stresses[b]	Low permeability soils - Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady State Seepage Conditions	Use effective stresses. Residual strengths should be used where previous shear deformation or sliding has occurred.	Estimated from field measurements of pore water pressures, hydrostatic pressure computations for no flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils - use effective stresses	Free draining soils - First stage computations (before drawdown) - steady-state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.
	Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.	Low permeability soils - First stage computations - steady-state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained strengths are used pore water pressures are set to zero.

[a] Effective stress parameters can be obtained from consolidated-drained (CD, S) tests (either direct shear or triaxial) or consolidated-undrained (CU, R) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the "R" or "total stress" envelope and associated c and ö, from CU, R tests should not be used.

[b] For saturated soils use ö = 0; total stress envelope with ö > 0 is only applicable to partially saturated soils.

6-6. Minimum Acceptable Factors of Safety

The minimum required safety factors for the preceding design conditions along with the portion of the embankment for which analyses are required and applicable shear test data are shown in Table 6-1b.

6-7. Measures to Increase Stability

Means for improving weak and compressible foundations to enable stable embankments to be constructed thereon are discussed in Chapter 7. Methods of improving embankment stability by changes in embankment section are presented in the following paragraphs.

a. Flatten embankment slopes. Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation type failure that takes place entirely within the embankment. Flattening embankment slopes reduces gravity forces tending to cause failure, and increases the length of potential failure surfaces (and therefore increases resistance to sliding).

Table 6-1b
Minimum Factors of Safety - Levee Slope Stability

	Applicable Stability Conditions and Required Factors of Safety			
Type of Slope	End-of-Construction	Long-Term (Steady Seepage)	Rapid Drawdown [a]	Earthquake[b]
New Levees	1.3	1.4	1.0 to 1.2	(see below)
Existing Levees	--	1.4[c]	1.0 to 1.2	(see below)
Other Embankments and dikes[d]	1.3[e,f]	1.4[e,f]	1.0 to 1.2[f]	(see below)

[a] Sudden drawdown analyses. F. S. = 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods preceding drawdown. F. S. = 1.2 applies to pool level, likely to persist for long periods prior to drawdown.
[b] See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.
[c] For existing slopes where either sliding or large deformation have occurred previously and back analyses have been performed to establish design shear strengths lower factors of safety may be used. In such cases probabilistic analyses may be useful in supporting the use of lower factors of safety for design.
[d] Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, river banks, and excavation slopes.
[e] Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. In such cases higher factors of safety may be required for end-of-construction to ensure stability during the time the excavation is to remain open. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.
[f] Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

b. Stability berms. Berms essentially provide the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing a substantial increase in the failure path. Thus, berms can be an effective means of stabilization not only for shallow foundation and embankment type failures but for more deep-seated foundation failures as well. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the embankment and prevent further movement.

6-8. Surface Slides

Experience indicates that shallow slides may occur in levee slopes after heavy rainfall. Failure generally occurs in very plastic clay slopes. They are probably the result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. These failures require maintenance and could be eliminated or reduced in frequency by using less plastic soils near the surface of the slopes or by chemical stabilization of the surface soils.

Section III
Settlement

6-9. General

Evaluation of the amount of postconstruction settlement that can occur from consolidation of both embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment. Many districts overbuild a levee by a given percent of its height to take into account anticipated settlement both of the foundation and within the levee fill itself. Common allowances are 0 to 5 percent for compacted fill, 5 to 10 percent for semicompacted fill, 15 percent for uncompacted fill, and 5 to 10 percent for hydraulic fill. Overbuilding does however increase the severity of stability problems and may be impracticable or undesirable for some foundations.

6-10. Settlement Analyses

Settlement estimates can be made by theoretical analysis as set forth in EM 1110-1-1904. Detailed settlement analyses should be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils. Where foundation and embankment soils are pervious or semipervious, most of the settlement will occur during construction. For impervious soils it is usually conservatively assumed that all the calculated settlement of a levee built by a normal sequence of construction operations will occur after construction. Where analyses indicate that more foundation settlement would occur than can be tolerated, partial or complete removal of compressible foundation material may be necessary from both stability and settlement viewpoints. When the depth of excavation required to accomplish this is too great for economical construction, other methods of control such as stage construction or vertical sand drains may have to be employed, although they seldom are justified for this purpose.

Chapter 7
Levee Construction

Section I
Levee Construction Methods

7-1. Classification of Methods

a. Levee embankments classified according to construction methods used are listed in Table 7-1 for levees composed of impervious and semipervious materials (i.e., those materials whose compaction characteristics are such as to produce a well-defined maximum density at a specific optimum water content). While the central portion of the embankment may be Category I (compacted) or II (semicompacted), riverside and landside berms (for seepage or stability purposes) may be constructed by Category II or III (uncompacted) methods.

b. Pervious levee fill consisting of sands or sands and gravels may be placed either in the dry with normal earthmoving equipment or by hydraulic fill methods. Except in seismically active areas or other areas requiring a high degree of compaction, compaction by vibratory means other than that afforded by tracked bulldozers is not generally necessary. Where underwater placement is required, it can best be accomplished with pervious fill using end-dumping, dragline, or hydraulic means, although fine-grained fill can be so placed if due consideration is given to the low density and strength obtained using such materials.

Section II
Foundations

7-2. Foundation Preparation and Treatment

a. General. Minimum foundation preparation for levees consists of clearing and grubbing, and most levees will also require some degree of stripping. Clearing, grubbing, stripping, the disposal of products therefrom, and final preparation are discussed in the following paragraphs.

b. Clearing. Clearing consists of complete removal of all objectional and/or obstructional matter above the ground surface. This includes all trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and similar debris. The entire foundation area under the levee and berms should be cleared well ahead of any following construction operations.

c. Grubbing. Grubbing consists of the removal, within the levee foundation area, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. Grubbing is usually not necessary beneath stability berms. Roots or other intrusions over 38.1 mm (1-1/2 in.) in diameter within the levee foundation area should be removed to a depth of 0.91 m (3 ft) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation area. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill, consisting or material similar to adjoining soils, should be placed in layers up to the final foundation grade and compacted to a density equal to the adjoining undisturbed material. This will avoid "soft spots" under the levee and maintain the continuity of the natural blanket.

d. Stripping. After foundation clearing and grubbing operations are complete, stripping is commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil. The depth of stripping

Table 7-1
Classification According to Construction Method of Levees Composed of Impervious and Semipervious Materials

Category	Construction Method	Use
I. Compacted	Specification of:	Provides embankment section occupying minimum space. Provides strong embankments of low compressibility needed adjacent to concrete structures or forming parts of highway systems.
	a. Water content range with respect to standard effort optimum water content	
	b. Loose lift thickness (152.4 mm to 228.6 mm (6-9 in.))	
	c. Compaction equipment (sheepsfoot or rubber-tired rollers)	Requires strong foundation of low compressibility and availability of borrow materials with natural water contents reasonably close to specified ranges.
	d. Number of passes to attain a given percent compaction based on standard maximum density	
	e. Minimum required density	Used where field inspection is not constant throughout the project.
II. Semicompacted	Compaction of fill materials at their natural water content (i.e., no water content control). Borrow materials known to be too wet would require some drying before placement. Placed in thicker lifts than Category I (about 304.8 mm (12 in.)) and compacted either by controlled movement of hauling and spreading equip ment or by fewer passes of sheepsfoot or rubber-tired rollers. Compaction evaluated relative to 15-blow compaction test.	The most common type of levee construction used in reaches where: *a.* There are no severe space limitations and steep-sloped Category I embankments are not required. *b.* Relatively weak foundations could not support steep-sloped Category I embankments. *c.* Underseepage conditions are such as to required wider embankment base than is provided by Category I construction. *d.* Water content of borrow materials or amount of rainfall during construction season is such as not to justify Category I compaction.
III. Uncompacted	*a.* Fill cast or dumped in place in thick layers with little or no spreading or compaction.	Levees infrequently constructed today using method except for temporary emergency. Both methods are used for construction of stability berms, pit fills and seepage berms.
	b. Hydraulic fill by dredge, often from channel excavation.	

is determined by local conditions and normally varies from 152.4 to 304.8 mm (6 to 12 in.) Stripping is usually limited to the foundation of the levee embankment proper, not being required under berms. All stripped material suitable for use as topsoil should be stockpiled for later use on the slopes of the embankment and berms. Unsuitable material must be disposed of by methods described in the next paragraph.

e. Disposal of debris. Debris from clearing, grubbing, and stripping operations can be disposed of by burning in areas where this is permitted. When burning is prohibited by local regulations, it needs to be disposed of in an environmentally approved manner.

f. Exploration trench. An exploration trench (often termed "inspection trench") should be excavated under all levees unless special conditions as discussed later warrant its omission. The purpose of this trench is to expose or intercept any undesirable underground features such as old drain tile, water or sewer lines, animal burrows, buried logs, pockets of unsuitable material, or other debris. The trench should be located at or near the centerline of hauled fill levees or at or near the riverside toe of sand levees so as to connect with waterside impervious facings. Dimensions of the trench will vary with soil conditions and embankment configurations. Backfill should be placed only after a careful inspection of the excavated trench to ensure that seepage channels or undesirable material are not present; if they are, they should be dug out with a base of sufficient width to allow backfill compaction with regular compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment will be required. Trenches should have a minimum depth of 1.83 m (6 ft) except for embankment heights less than 1.83 m (6 ft), in which case the minimum depth should equal the embankment height. Exploration trenches can be omitted where landside toe drains beneath the levee proper constructed to comparable depths are employed (toe drains are discussed in more detail later in this chapter).

g. Dewatering. Dewatering levee foundations for the purpose of excavation and back filling in the dry is expensive if more than simple ditches and sumps are required, and is usually avoided if at all possible. The cost factor may be an overriding consideration in choosing seepage control measures other than a compacted cutoff trench, such as berms, blankets, or relief wells. Where a compacted cutoff trench involving excavation below the water table must be provided, dewatering is essential. TM 5-818-5 provides guidance in dewatering system design.

h. Final foundation preparation. Soft or organic spots in the levee foundation should be removed and replaced with compacted material. Except in special cases where foundation surfaces are adversely affected by remolding (soft foundations for instance), the foundation surface upon or against which fill is to be placed should be thoroughly broken up to a depth of at least 152.4 mm (6 in.) prior to the placement of the first lift of fill. This helps to ensure good bond between the foundation and fill and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to avoid saturation from rainfall.

7-3. Methods of Improving Stability

a. General. Levees located on foundation soils that cannot support the levee embankment because of inadequate shear strength require some type of foundation treatment if the levee is to be built. Foundation deposits that are prone to cause problems are broadly classified as follows: (1) very soft clays, (2) sensitive clays, (3) loose sands, (4) natural organic deposits, and (5) debris deposited by man. Very soft clays are susceptible to shear failure, failure by spreading, and excessive settlement. Sometimes soft clay deposits have a zone of stronger clay at the surface, caused by dessication, which if strong enough may eliminate the need for expensive treatment. Sensitive clays are brittle and even though possessing considerable strength in the undisturbed state, are subject to partial or complete loss of strength upon disturbance. Fortunately, extremely sensitive clays are rare. Loose sands are also sensitive to disturbance and can liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of slopes. Most organic soils are very compressible and exhibit low shear strength. The physical characteristics and behavior of organic deposits such as peat can sometimes be predicted with some degree of accuracy. Highly fibrous organic soils with water contents of 500 percent or more generally consolidate and gain strength rapidly. The behavior of debris deposited by man, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict. The following paragraphs discuss methods of dealing with foundations that are inadequate for construction of proposed levees.

b. Excavation and replacement. The most positive method of dealing with excessively compressible and/or weak foundation soils is to remove them and backfill the excavation with suitable compacted material. This procedure is feasible only where deposits of unsuitable material are not excessively deep. Excavation and replacement should be used wherever economically feasible.

c. Displacement by end dumping.

(l) Frequently low levees must be constructed across sloughs and stream channels whose bottoms consist of very soft fine-grained soils (often having high organic content). Although the depths of such deposits may not be large, the cost of removing them may not be justified, as a levee of adequate stability can be obtained by end-dumping fill from one side of the slough or channel, pushing the fill over onto the soft materials, and continually building up the fill until its weight displaces the foundation soils to the sides and front. By continuing this operation, the levee can finally be brought to grade. The fill should be advanced with a V-shaped leading edge so that the center of the fill is most advanced, thereby displacing the soft material to both sides. A wave of displaced foundation material will develop (usually visible) along the sides of the fill and should not be removed. A disadvantage of this method is that all soft material may not be displaced which could result in slides as the embankment is brought up and/or differential settlement after construction. Since this type of construction produces essentially uncompacted fill, the design of the levee section should take this into account.

(2) When this method of foundation treatment is being considered for a long reach of levee over unstable areas such as swamps, the possibility of facilitating displacement by blasting methods should be evaluated. Blasters' Handbook (1966) (Appendix A-2) presents general information on methods of blasting used to displace soft materials.

(3) The end-dumping method is also used to provide a working platform on soft foundation soils upon which construction equipment can operate to construct a low levee. In this case, only enough fill material is hauled in and dozed onto the foundation to build a working platform or pad upon which the levee proper can be built by conventional equipment and methods. Material forming the working platform should not be stockpiled on the platform or a shear failure may result. Only small dozers should be used to spread and work the material. Where the foundation is extremely weak, it may be necessary to use a small clamshell to spread the material by casting it over the area.

d. Stage construction.

(1) General. Stage construction refers to the building of an embankment in stages or intervals of time. This method is used where the strength of the foundation material is inadequate to support the entire weight of the embankment, if built continuously at a pace faster than the foundation material can drain. Using this method, the embankment is built to intermediate grades and allowed to rest for a time before-placing more fill. Such rest periods permit dissipation of pore water pressures which results in a gain in strength so that higher embankment loadings may be supported. Obviously this method is appropriate when pore water

pressure dissipation is reasonably rapid because of foundation stratification resulting in shorter drainage paths. This procedure works well for clay deposits interspersed with highly pervious silt or sand seams. However, such seams must have exits for the escaping water otherwise they themselves will become seats of high pore water pressure and low strengths (pressure relief wells can be installed on the landside to increase the efficiency of pervious layers in foundation clays). Initial estimates of the time required for the needed strength gain can be made from results of consolidation tests and study of boring data. Piezometers should be installed during construction to monitor the rate of pore water dissipation, and the resumption and rate of fill placement should be based on these observations, together with direct observations of fill and foundation behavior. Disadvantages of this method are the delays in construction operation, and uncertainty as to its scheduling and efficiency.

(2) Prefabricated vertical (wick) drains. If the expected rate of consolidation under stage construction is unacceptably slow, it may be increased by the use of prefabricated vertical (wick) drains. Such drains are geotextile wrapped plastic cores that provide open flowage areas in the compressible stratum. Their purpose is to reduce the length of drainage paths, thus speeding up primary consolidation. The wick drains are very thin and about 101.6 mm (4 in.) wide. They can be pushed into place through soft soils over 30.5 m (100 ft) deep. Before the drains are installed, a sand drainage blanket is placed on the foundation which serves not only to tie the drains together and provide an exit for escaping pore water, but as a working platform as well. This drainage blanket should not continue across the entire base width of the embankment, but should be interrupted beneath the center.

e. Densification of loose sands. The possibility of liquefaction of loose sand deposits in levee foundations may have to be considered. Since methods for densifying sands, such as vibroflotation, are costly, they are generally not considered except in locations of important structures in a levee system. Therefore, defensive design features in the levee section should be provided, such as wider levee crest, and flatter slopes.

Section III
Embankments

7-4. Embankment Construction Control

a. Construction control of levees may present somewhat different problems from that of dams because:

(1) Construction operations may be carried on concurrently along many miles of levee, whereas the majority of dams are less than about 0.8 km (0.5 mile) in length and only in a few cases are dams longer than 4.8 km (3 miles). This means that more time is needed to cover the operations on many levee jobs.

(2) While inspection staff and testing facilities are located at the damsite, levee inspection personnel generally operate out of an area office which may be a considerable distance from the levee project.

(3) There are frequently fiscal restraints which prevent assigning an optimum number of inspectors on levee work or even one full-time inspector on small projects. Under these conditions, the inspectors used must be well-trained to observe construction operations, minimizing the number of field density tests in favor of devoting more time to visual observations, simple measurements, and expedient techniques of classifying soils, evaluating the suitability of their water content, observing behavior of construction equipment on the fill, and indirectly assessing compacted field densities.

b. Although it has previously been stated that only limited foundation exploration and embankment design studies are generally needed in areas where levee heights are low and foundation conditions adequate (i.e., no question of levee stability), the need for careful construction control by competent inspection exists as well as at those reaches where comprehensive investigations and analyses have been made. Some of the things that can happen during construction that can cause failure or distress of even low embankments on good foundations are given in Table 7-2.

Table 7-2
Embankment Construction Deficiencies

Deficiency	Possible Consequences
Organic material not stripped from foundation	Differential settlements; shear failure; internal erosion caused by through seepage
Highly organic or excessively wet or dry fill	Excessive settlements; inadequate strength
Placement of pervious layers extending completely through the embankment	Allows unimpeded through seepage which may lead to internal erosion and failure
Inadequate compaction of embankment (lifts too thick, haphazard coverage by compacting equipment, etc.)	Excessive settlements; inadequate strength; through seepage
Inadequate compaction of backfill around structures in embankment	Excessive settlements; inadequate strength; provides seepage path between structure and material which may lead to internal erosion and failure by piping

7-5. Embankment Zoning

As a general rule levee embankments are constructed as homogeneous sections because zoning is usually neither necessary nor practicable. However, where materials of varying permeabilities are encountered in borrow areas, the more impervious materials should be placed toward the riverside of the embankment and the more pervious material toward the landside slope. Where required to improve underseepage conditions, landside berms should be constructed of the most pervious material available and riverside berms of the more impervious materials. Where impervious materials are scarce, and the major portion of the embankment must be built of pervious material, a central impervious core can be specified or, as is more often done, the riverside slope of the embankment can be covered with a thick layer of impervious material. The latter is generally more economical than a central impervious core and, in most cases, is entirely adequate.

7-6. Protection of Riverside Slopes

a. The protection needed on a riverside slope to withstand the erosional forces of waves and stream currents will vary, depending on a number of factors:

(1) The length of time that floodwaters are expected to act against a levee. If this period is brief, with water levels against the levee continually changing, grass protection may be adequate, but better protection may be required if currents or waves act against the levee over a longer period.

(2) The relative susceptibility of the embankment materials to erosion. Fine-grained soils of low plasticity (or silts) are most erodible, while fat clays are the least erodible.

(3) The riverside slope may be shielded from severe wave attack and currents by timber stands and wide space between the riverbank and the levee.

(4) Structures riverside of the levee. Bridge abutments and piers, gate structures, ramps, and drainage outlets may constrict flow and cause turbulence with resultant scour.

(5) Turbulence and susceptibility to scour may result if levee alignment includes short-radius bends or if smooth transitions are not provided where levees meet high ground or structures.

(6) Requirements for slope protection are reduced when riverside levee slopes are very flat as may be the case for levees on soft foundations. Several types of slope protection have been used including grass cover, gravel, sand-asphalt paving, concrete paving, articulated concrete mat, and riprap, the choice depending upon the degree of protection needed and relative costs of the types providing adequate protection.

b. Performance data on existing slopes under expected conditions as discussed above are invaluable in providing guidance for the selection of the type of slope protection to be used.

c. Sometimes it may be concluded that low cost protection, such as grass cover, will be adequate in general for a levee reach, but with a realization that there may be limited areas where the need for greater protection may develop under infrequent circumstances. If the chances of serious damage to the levee in such areas are remote, good engineering practice would be to provide such increased protection only if and when actual problems develop. Of course, it must be possible to accomplish this expeditiously so that the situation will not get out of hand. In any event, high-class slope protection, such as riprap, articulated mat, or paving should be provided on riverside slopes at the following locations:

(1) Beneath bridges, since adequate turf cannot be generally established because of inadequate sunlight.

(2) Adjacent to structures passing through levee embankments.

d. Riprap is more commonly used than other types of revetments when greater protection than that afforded by grass cover is required because of the relative ease of handling, stockpiling, placement, and maintenance. Guidance on the design of riprap revetment to protect slopes against currents is presented in EM 1110-2-1601. Where slopes are composed of erodible granular soils or fine-grained soils of low plasticity, a bedding layer of sand and gravel or spalls, or plastic filter cloth should be provided beneath the riprap.

e. When suitable rock is not available within economical haul distances, soil cement may provide the most economical slope protection (see Appendix G).

Chapter 8
Special Features

Section I
Pipelines and Other Utility Lines Crossing Levees

8-1. General Considerations

a. Serious damage to levees can be caused by inadequately designed or constructed pipelines, utility conduits, or culverts (all hereafter referred to as "pipes") beneath or within levees. Each pipe crossing should be evaluated for its potential damage which would negatively impact the integrity of the flood protection system and could ultimately lead to catastrophic failure. During high water, seepage tends to concentrate along the outer surface of pipes resulting in piping of fill or foundation material. High water also results in uplift pressures that may cause buoyancy of some structures. Seepage may also occur because of leakage from the pipe. In the case of pipes crossing over levees, leakage can cause erosion in the slopes. In addition, loss of fill or foundation material into the pipe can occur if joints are open. The methods of pipe installation should be understood by the designer to anticipate problems that may occur. Some of the principal inadequacies that are to be avoided or corrected are as follows:

(1) Pipes having inadequate strength to withstand loads of overlying fill or stresses applied by traffic.

(2) Pipe joints unable to accommodate movements resulting from foundation or fill settlement.

(3) Unsuitable backfill materials or inadequately compacted backfill.

(4) High pressures from directional drilling that could result in hydro-fracturing the surrounding materials.

b. Some state and local laws prohibit pipes from passing through or under certain categories of levees. As a general rule, this should not be done anyway, particularly in the case of pressure lines. However, since each installation is unique, pipes in some instances may be allowed within the levee or foundation. Major factors to be considered in deciding if an existing pipe can remain in place under a new levee or must be rerouted over the levee, or if a new pipe should be laid through or over the levee are as follows:

(1) The height of the levee.

(2) The duration and frequency of high water stages against the levee.

(3) The susceptibility to piping and settlement of levee and foundation soils.

(4) The type of pipeline (low or high pressure line, or gravity drainage line).

(5) The structural adequacy of existing pipe and pipe joints, and the adequacy of the backfill compaction.

(6) The feasibility of providing closure in event of ruptured pressure lines, or in the event of failure of flap valves in gravity lines during high water.

(7) The ease and frequency of required maintenance.

(8) The cost of acceptable alternative systems.

(9) Possible consequences of piping or failure of the pipe.

(10) Previous experience with the owner in constructing and maintaining pipelines.

General criteria for pipes crossing levees are given in Table 8-1.

Table 8-1
Criteria for Pipelines Crossing Levees

Pipelines	Leaving Existing Pipeline in Foundations of Proposed Levees	New Pipeline Installation: Pipes Through Levees	New Pipeline Installation: Pipes Over Levees
Must be known to be in good condition	X		
Must have adequate strength to withstand levee loading	X	X	
Must have adequate cover as needed to prevent damage by vehicular traffic or heavy equipment			X
Must have adequate cover for frost protection			X
Must have sufficient flexibility in joints to adjust under expected settlement and stretching of pipe	X	X	X
Pressure lines must have provisions for rapid closure in event of leakage or rupture	X	X	X
Gravity discharge pipes must have provisions for emergency closure in event of inoperative flap valves on riverside end	X	X	
Must have pervious backfill under landside third of levee where:			
a. Foundation materials are susceptible to piping	X		
b. Levee materials are susceptible to piping		X	

8-2. General Considerations for Pipelines Crossing Through or Under Levees

a. General. As has been noted previously, it is preferable for all pipes to cross over a levee rather than penetrate the embankment or foundation materials. This is particularly true for pipes carrying gas or fluid under pressure. Before consideration is given to allowing a pressure pipe (and possibly other types of pipe) to extend through or beneath the levee, the pipe owner should provide an engineering study to support his request for such installation. The owner, regardless of the type of pipe, should show adequate capability to properly construct and/or maintain the pipe. Future maintenance of pipe by the owner must be carefully evaluated. It may be necessary to form an agreement to the effect that should repairs to a pipe in the levee become necessary, the pipe will be abandoned, sealed, and relocated over the levee.

b. Existing pipes

(1) All existing pipelines must be located prior to initiation of embankment construction. As previously noted, inspection trenches may reveal abandoned pipes not on record. It is preferable that all abandoned pipes be removed during grubbing operations and the voids backfilled. Any existing pipe should meet or be made to meet the criteria given in Table 8-1. If this is not feasible and removal is not practical, they should be sealed, preferably by completely filling them with concrete. Sealed pipes must also meet the criteria given in Table 8-1 relating to prevention of seepage problems.

(2) In general, existing pressure pipes should be relocated over the proposed new levee. Rupture or leakage from such pipes beneath a levee produces extremely high gradients that can have devastating effects on the integrity of the foundation. Therefore, as indicated by the criteria in Table 8-1, it is imperative that pressure pipes be fitted with rapid closure valves or devices to prevent escaping gas or fluid from damaging the foundation.

(3) Although gravity drainage lines may be allowed or even required after the levee is completed, it is likely that existing pipes will not have sufficient strength to support the additional load induced by the embankment. Therefore, existing pipes must be carefully evaluated to determine their supporting capacity before allowing their use in conjunction with the new levee.

c. New Pipelines. Generally, the only new pipelines allowed to penetrate the foundation or embankment of the levee are gravity drainage lines. The number of gravity drainage structures should be kept to an absolute minimum. The number and size of drainage pipes can be reduced by using such techniques as ponding to reduce the required pipe capacity.

8-3. General Considerations for Pipelines Crossing Over Levees

In the past the term and concept of freeboard was used to account for hydraulic, geotechnical, construction, operation and maintenance uncertainties. Pipelines crossing over the levee were encouraged to be within the freeboard zone to reduce or eliminate many of the dangers that are inherent with pipelines crossing through the embankment or foundation. The term and concept of freeboard to account for these uncertainties is no longer used in the design of levee projects. Therefore, since freeboard no longer exists, pipes must cross over the completed levee cross section. Problems do exist, however, with pipelines crossing over the levee. These pipes must be properly designed and constructed to prevent (a) flotation if submerged, (b) scouring or erosion of the embankment slopes from leakage or currents, and (c) damage from debris carried by currents, etc. In some areas climatic conditions will require special design features. Guidance on design methods and construction practices will be given later in this chapter.

8-4. Pipe Selection

a. EM 1110-2-2902 contains a discussion of the advantages and disadvantages of various types of pipe (i.e., corrugated metal, concrete, cast iron, steel, clay, etc.). The selection of a type of pipe is largely dependent upon the substance it is to carry, its performance under the given loading, including expected deflections or settlement, and economy. Although economy must certainly be considered, the overriding factor must be safety, particularly where urban levees are concerned.

b. The earth load acting on a pipe should be determined as outlined in EM 1110-2-2902. Consideration must also be given to live loads imposed from equipment during construction and the loads from traffic and maintenance equipment after the levee is completed. The respective pipe manufacturers organizations have recommended procedures for accounting for such live loads. These recommended procedures should be followed unless the pipe or roadway owners have more stringent requirements.

c. Required strengths for standard commercially available pipe should be determined by the methods recommended by the respective pipe manufacturers organizations. Where cast-in-place pipes are used, design procedures outlined in EM 1110-2-2902 should be followed. Abrasion and corrosion of corrugated steel pipe should be accounted for in design using the method given in Federal Specification WW-P-405B(1) (Appendix A) for the desired design life. The design life of a pipe is the length of time it will be in service without requiring repairs. The term does not imply the pipe will fail at the end of that time. Normally, a design life of 50 years can be economically justified. Corrugated pipe should always be galvanized and protected by a bituminous or other acceptable coating as outlined in EM 1110-2-2902. Protective coatings may be considered in determining the design life of a pipe.

d. Leakage from or infiltration into any pipe crossing over, through, or beneath a levee must be prevented. Therefore, the pipe joints as well as the pipe itself must be watertight. For pipes located within or beneath the embankment, the expected settlement and outward movement of the soil mass must be considered. Where considerable settlement is likely to occur the pipe should be cambered (para 8-7). Generally, flexible corrugated metal pipes are preferable for gravity lines where considerable settlement is expected. Corrugated metal pipe sections should be joined by exterior coupling bands with a gasket to assure watertightness. Where a concrete pipe is required and considerable settlement is anticipated, a pressure-type joint with concrete alignment collars should be used. The collars must be designed either to resist or accommodate differential movement without losing watertight integrity. Where settlement is not significant, pressure-type joints capable of accommodating minor differential movement are sufficient. Design details for concrete collars are shown in EM 1110-2-2902. Cast iron and steel pipes should be fitted with flexible bolted joints. Steel pipe sections may be welded together to form a continuous conduit. All pressure pipes should be pressure tested at the maximum anticipated pressure before they are covered and put into use.

e. During the design, the potential for electrochemical or chemical reactions between the substratum materials or groundwater and construction materials should be determined. If it is determined that there will be a reaction, then the pipe and/or pipe couplings should be protected. The protective measures to be taken may include the use of cathodic protection, coating of the pipe, or use of a corrosion-resistant pipe material.

8-5. Antiseepage Devices

a. Antiseepage devices have been employed in the past to prevent piping or erosion along the outside wall of the pipe. The term "antiseepage devices" usually referred to metal diaphragms (seepage fins) or concrete collars that extended from the pipe into the backfill material. The diaphragms and collars were often referred to as "seepage rings." However, many piping failures have occurred in the past where seepage rings were used. Assessment of these failures indicated that the presence of seepage rings often results in poorly compacted backfill at its contact with the structure.

b. Where pipes or conduits are to be constructed through new or existing levees:

(1) Seepage rings or collars should not be provided for the purpose of increasing seepage resistance. Except as provided herein, such features should only be included as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations. When needed for these purposes, collars with a minimum projection from the pipe surface should be used.

(2) A 0.45-m (18-in.) annular thickness of drainage fill should be provided around the landside third of the pipe, regardless of the size and type of pipe to be used, where landside levee zoning does not provide for such drainage fill. For pipe installations within the levee foundation, the 0.45-m (18-in.) annular thickness of drainage fill shall also be provided, to include a landside outlet through a blind drain to ground surface at the levee toe, connection with pervious underseepage features, or through an annular drainage fill outlet to ground surface around a manhole structure. Figure 8-1 shows typical sections of drainage structures through levees. Figure 8-2 shows typical precast conduits through the levee.

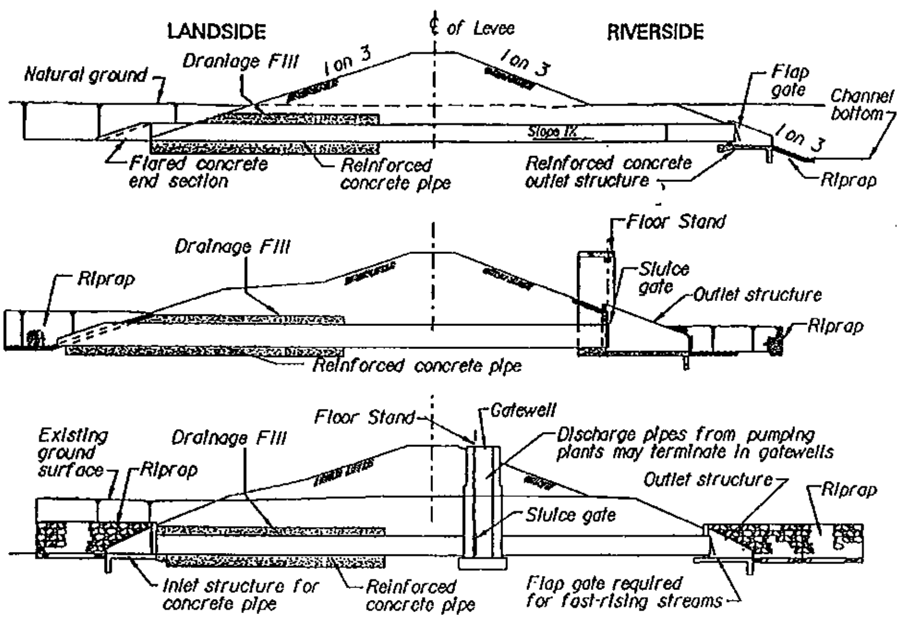


Figure 8-1. Typical sections, drainage structures through levees

8-6. Closure Devices

a. All pipes allowed to penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. Gravity lines should be provided with flap-type or slide-type service gates on the riverside of the levee. Automatic flap-type gates are usually used where the water is likely to rise to the "Gate Closing Stage" rather suddenly and where the water stage is likely to fluctuate within a few feet above and below the "Gate Closing Stage" for prolonged periods of time during flood season. Automatic gates are also required on slower rising streams or bodies of water where frequent visits from operating personnel are not practical.

b. Slide-type gates are usually preferred as service gates where the rate of rise of the water during major floods is slow, enough (minimum of 12-hr flood prediction time) to give ample time for safe operation. The principal advantages of the slide gate in comparison with automatic flap gates are greater reliability of operation and the ease with which emergency closure can be made in event obstructions prevent closure of the gate. Usually emergency closure can be made by filling the manhole with sandbags. The obvious

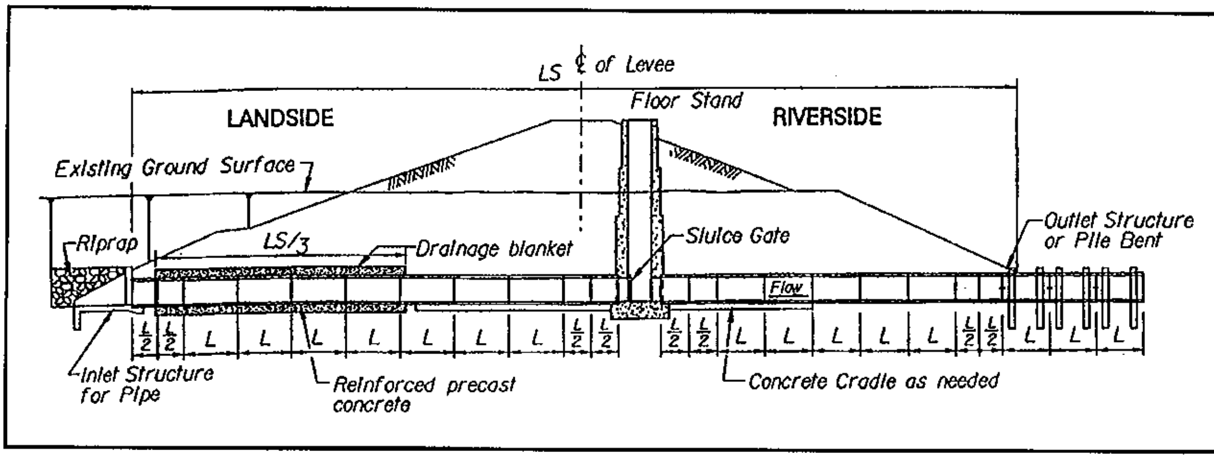


Figure 8-2. Typical precast conduit (levees)

disadvantage of slide type gates is that personnel must be on hand for their operation. Also their initial cost is generally greater than that for a flap-type gate.

c. A slide-type gate with a flap-type gate attachment is often used and affords the advantages of automatic flap gate operation with the added safety of the slide-type gate. Such installations usually eliminate the need for a supplemental emergency gate as described below.

d. Experience has shown that service gates occasionally fail to close completely during critical flood periods because of clogging by debris, mechanical malfunctions, or other causes. This, of course, can cause flooding of, the protected areas. Supplemental emergency gates are intended to minimize these risks insofar as necessary and economically practical. For an emergency gate to be effective it must be located so that its controls are accessible during flood stage. Provisions required for emergency protection of other areas should be consistent with the risks and cost involved.

e. Pressure pipes should be fitted with valves at various stations that can be closed rapidly to prevent gas or fluid from escaping within or beneath a levee should the pipe rupture within these areas. Provisions for closure of pressure pipes on the water side must also be provided to prevent backflow of floodwater into the protected area should the pipe rupture. These requirements should generally be followed in other areas, but may be relaxed to be consistent with the risks and costs involved.

8-7. Camber

The alignment of a gravity structure must be such as to provide for a continuous slope toward the outlet. Settlement of the embankment and foundation can significantly alter the initial grade line of a pipe. Therefore, the expected settlement of the levee must be considered in establishing the initial grade line. If the settlement will result in an upward gradient in the direction of flow or not allow the desired gradient to be maintained, the pipe should be cambered. The amount of camber required can usually be taken as the mirror image of the settlement curve along a line established by the final required grade. The camber should then be laid out, preferably as a vertical curve, on a grade such that all parts of the pipe will slope toward the outlet when installed. If the gradient of the pipe is limited and the camber will initially result in a slope away from the outlet, the portion of the pipe from the inlet up to the point of greatest load may be installed level. The remaining portion of the pipe is then installed on a vertical curve tangent to the first portion of the pipe.

Regardless of the type of pipe selected, movements at the joints must be considered as discussed in paragraph 8-4d.

8-8. Installation Requirements

a. General. The installation of pipes or other structures within the levee or foundation probably requires the greatest care and the closest supervision and inspection of any aspect of levee construction. Most failures of levee systems have initiated at the soil-structure interface and therefore every effort must be made to ensure that these areas are not susceptible to piping. Of overriding importance is good compaction of the backfill material along the structure. Pipes installed by open trench excavation should be installed in the dry and a dewatering system should be used where necessary. Pipes installed by directional drilling, microtunneling, or other trenchless methods require special consideration.

b. Pipes crossing through or beneath levees

(1) The preferred method of installing pipes within the embankment or foundation of a levee has historically been by the open cut method. Preferably, new levees should be brought to a grade about 610.8 mm (2 ft) above the crown of the pipe. This allows the soil to be preconsolidated before excavating the trench. The trench should be excavated to a depth of about 610.8 mm (2 ft) below the bottom of the pipe and at least 1.2 m (4 ft) wider than the pipe. The excavated material should be selectively stockpiled so that it can be replaced in a manner that will not alter the embankment zoning if there is some or will result in the more impervious soils on the riverside of the levee.

(2) After the trench has been excavated, it should be backfilled to the pipe invert elevation. In impervious zones, the backfill material should be compacted with mechanical compactors to 95 percent standard density at about optimum water content.

(3) First-class bedding should be used for concrete pipe and other rigid pipe, as shown in EM 1110-2-2902 except no granular bedding should be used in impervious zones. For flexible pipe, the trench bottom should be flat to permit thorough tamping of backfill under the haunches of the pipe. Backfill should be compacted to 95 percent standard density at about optimum water content. The backfill should be brought up evenly on both sides of the pipe to avoid unequal side loads that could fail or move the pipe. Special care must be taken in the vicinity of any protrusions such as joint collars to ensure proper compaction. Where granular filter material is required, it should be compacted to a minimum of 80 percent relative density. In areas where backfill compaction is difficult to achieve, flowable, low strength concrete fill has been used to encapsulate pipes in narrow trenches.

(4) In existing levees, the excavation slopes should be stable, meet OSHA criteria, but in no case be steeper than 1V on 1H. The excavated material should be selectively stockpiled as was described for new levees. The pipe is installed as described in the previous paragraphs. Impervious material within 0.61 m (2 ft) of the pipe walls should be compacted to 95 percent standard density at optimum water content, with the remainder of the backfill placed at the density and water content of the existing embankment.

(5) Installation of pipes in existing levees by directional drilling, microtunneling, tunneling or jacking may be considered. It is recognized, that in some instances, installation by the open cut method is not feasible or cannot be economically justified. Where trenchless methods are allowed, special considerations are required.

(6) Pipes under levees.

(a) General. Pipes crossing beneath levees also require special considerations. Such crossings should be designed by qualified geotechnical engineers. Pipes constructed with open excavation methods should proceed in accordance with the requirements stated in the above paragraph, Pipes Crossing Through or Beneath Levees. If directional drilling or other trenchless methods are used, seepage conditions may be aggravated by the collapse of levee foundation material into the annular void between the bore and pipe. Penetration through the top stratum of fine-grained materials may concentrate seepage at those locations. Pipes constructed with trenchless methods should proceed only after a comprehensive evaluation of the following: comprehensive understanding of the subsurface soil and groundwater conditions to a minimum depth of 6.1 m (20 ft) below the lowest pipe elevation, locations of the pipe penetration entry and exit, construction procedure, allowable uplift pressures, on-site quality control and quality assurance monitoring during construction operation, grouting of the pipe annulus, backfilling of any excavated areas, and repair and reinstatement of the construction-staging areas. Guidance for construction of pipelines beneath levees using directional drilling is provided in Appendix A of WES CPAR-GL-98-1 (Staheli, et al. 1998). Guidance for construction of pipelines using microtunneling methods is provided in WES CPAR-GL-95-2 (Bennett, et al. 1995).

(b) Pipes installed by directional drilling. The pipe entry or exit location, when located on the protected (land) side, should be set back sufficiently from the land side levee toe to ensure that the pipe penetrates some depth of a pervious sand stratum but is no less than 91.5 m (300 ft) from the centerline of the levee crest. The pipe entry or exit location, when located on the unprotected (river) side, should be located at least 6.1 m (20 ft) riverward of the levee stability control line. This is the distance between the river side levee toe and an eroding bank line which will maintain the minimum design criteria for slope stability.

If directional drilling is to be used, the depth of the pipe under the levee should be at a level to maintain an adequate factor of safety against uplift from the pressurized drilling fluid during the drilling operation. A positive means of maintaining an open vent to the surface should be required whether through bored holes or downhole means while installing the drill pipe.

The drilling fluid should consist of a noncolloidal lubricating admixture to ensure suspension and removal of drilling cuttings. The pilot hole should be advanced at a rate to maintain a continuous return flow. The annular space should be sufficient to ensure that no blockage occurs with the drilling cuttings. The prereamer boring diameter should be of sufficient size to ensure that the production pipe can be advanced without delay and undue stress to the surrounding soils. The prereamer boring operation should be continuous for the down-slope and up-slope cutting segments. Excessive drilling fluid pressures can hydraulically fracture the levee foundation and levee embankment and should be avoided.

Where economically feasible, the pipeline should be bored through rock where the pipeline crosses the levee centerline.

The maximum allowable mud pressure acting against the borehole wall should be evaluated using the Delft equation presented in the Appendix A of WES CPAR-GL-98-1 (Staehli, et al., 1998). During construction, the actual mud pressure existing in the borehole must be measured by a pressure measuring device located on the outside of the drill string no more than 5 ft from the drill bit. The drilling operator should be required to monitor these pressures and adjust the drilling mud pressure so as not to exceed the maximum pressure determined by Delft equation.

Where the casing pipe is carrying multiple fibre optic cables and each cable is installed within its own HDPE inner duct, the detail shown in Figure 8-3a should be used to prevent preferred seepage path (both external and internal). The casing pipe must end in the encasements.

The directional drilling contract should be required to show proof that all of his pressure sensors and readout devices have been calibrated by a national standard within the last 6 months.

A full time inspector, not on directional drilling contractor's payroll, should be required to observe the construction.

The drilling fluid should be processed through an active drilling mud conditioning unit to remove the cuttings from the drill fluid and maintain its viscosity.

c. Pipes crossing over levees. Pipe crossings on the surface of the levee should be designed to counteract uplift of the empty pipe at the design high water stage. This may be accomplished by soil cover, anchors, headwalls, etc. All pipes on the riverside of the levee should have a minimum of 305 mm (1 ft) of soil cover for protection from debris during high water. It is desirable for pipe on the landward side to also be covered with soil. Pipes crossing beneath the levee crown should be provided with sufficient cover to withstand vehicular traffic as outlined in paragraph 8-4b. Depth of cover should also be at least the depth of local frost protection. Where mounding of soil over the pipe is required, the slope should be gentle to allow mowing equipment or other maintenance equipment to operate safely on the slopes. The approach ramps on the levee crown should not exceed 1V on 10H in order to allow traffic to move safely on the crown. The trenching details for pipelines cross-up and over-levees are shown in Figure 8-3b and Figure 8-3c.

Section II
Access Roads and Ramps

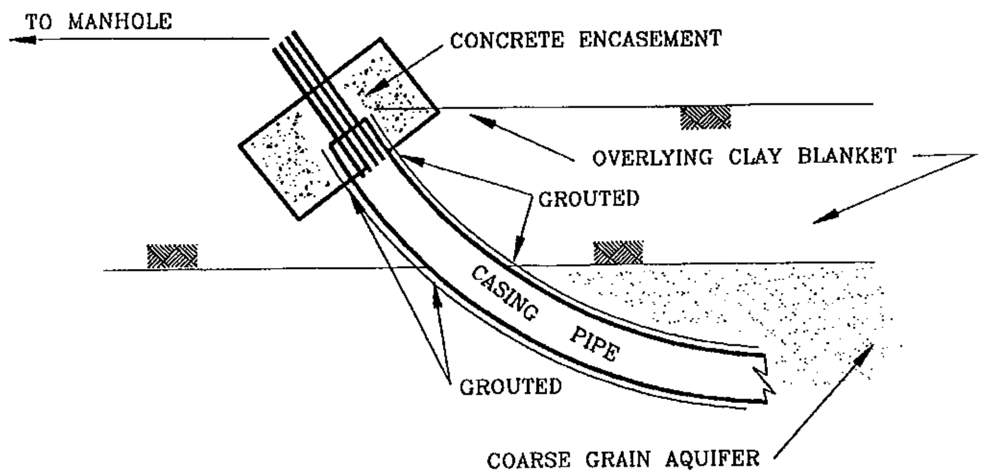
8-9. Access Roads

a. Access road to levee. Access roads should be provided to levees at reasonably close intervals in cooperation with state and local authorities. These roads should be all-weather roads that will allow access for the purpose of inspection, maintenance, and flood-fighting operations.

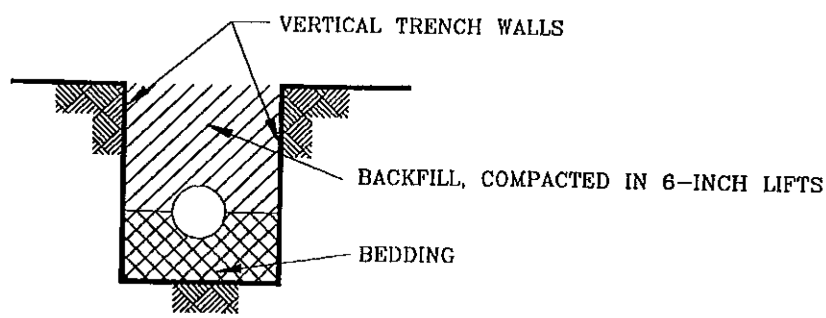
b. Access road on levee. Access roads, sometimes referred to as patrol roads, should be provided also on top of the levees for the general purpose of inspection, maintenance, and flood-fighting operations. This type of road should be surfaced with a suitable gravel or crushed stone base course that will permit vehicle access during wet weather without causing detrimental effects to the levee or presenting safety hazards to the levee inspection and maintenance personnel. The width of the road surfacing will depend upon the crown width of the levee, where roadway additions to the crown are not being used, and upon the function of the roadway in accommodating either one- or two-way traffic. On levees where county or state highways will occupy the crown, the type of surfacing and surfacing width should be in accordance with applicable county or state standards. The decision as to whether the access road is to be opened to public use is to be made by the local levee agency which owns and maintains the levee.

(1) Turnouts. Turnouts should be used to provide a means for the passing of two motor vehicles on a one-lane access road on the levee. Turnouts should be provided at intervals of approximately 762 m (2500 ft), provided there are no ramps within the reach. The exact locations of the turnouts will be dependent upon various factors such as sight distance, property lines, levee alignment, and desires of local interests. An example turnout for a levee with a 3.65 m (12-ft) levee crown is shown in Figure 8-4.

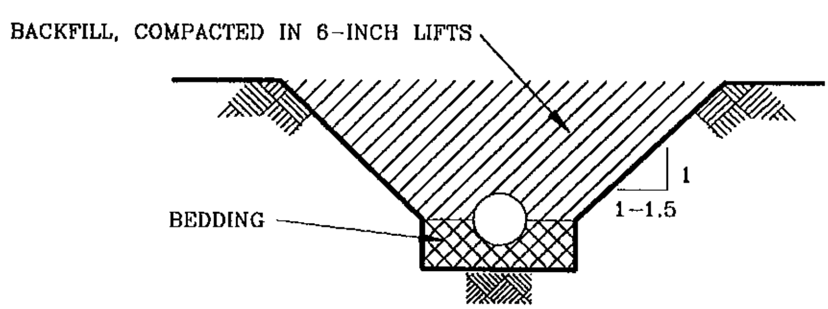
(2) Turnarounds. Turnarounds should be provided to allow vehicles to reverse their direction on all levees where the levee deadends, and no ramp exists in the vicinity of the deadend. An example turnaround for a levee with a 3.65-m (12-ft) crown is shown in Figure 8-5.



a. Detail of end casing (for pipe lines beneath the levees)



b. On riverside, within 50 ft of levee (for pipe lines cross-up and over-levees)



c. On landside (for pipelines cross-up and over-levees)

Figure 8-3. Details of pipeline levee crossing

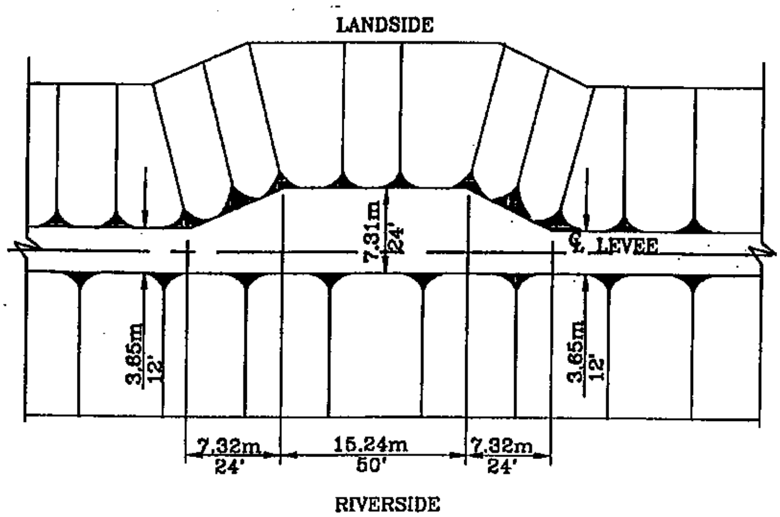


Figure 8-4. Example of levee turnout

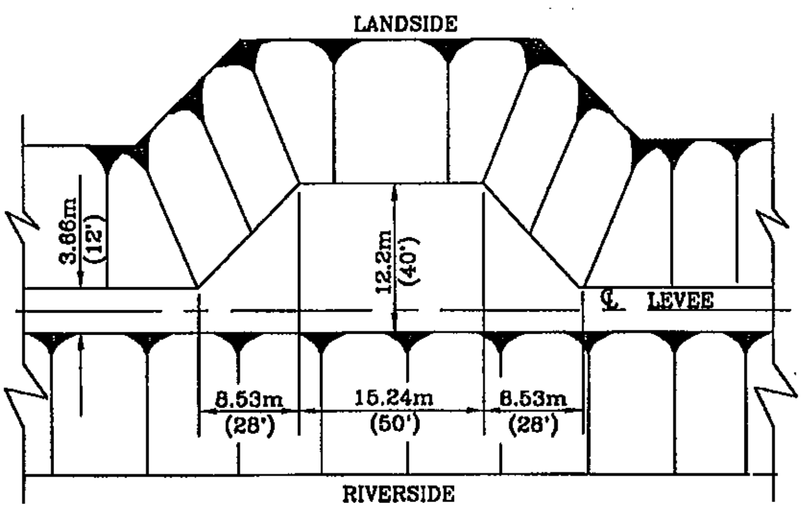


Figure 8-5. Example of levee turnaround

8-10. Ramps

a. Ramps should be provided at sufficient locations to permit vehicular traffic to access onto and from the levee. Ramps may be located on both the landside and the riverside of the levee. Ramps on the landside of the levee are provided to connect access roads leading to a levee with access roads on top of a levee and at other convenient locations to serve landowners who have property bordering the levee. Ramps are also provided on some occasions on the riverside of the levee to connect the access road on top of the levee with existing levee traverses where necessary. The actual locations of the ramps should have the approval of the local levee agency which owns and maintains the levee. When used on the riverside of the levee, they should be oriented to minimize turbulence during high water.

b. Ramps are classified as public or private in accordance with their function. Public ramps are designed to satisfy the requirements of the levee owner: state, county, township, or road district. Private ramps are usually designed with less stringent requirements and maximum economy in mind. Side-approach ramps should be used instead of right angle road ramps because of significant savings in embankment. The width of the ramp will depend upon the intended function. Some widening of the crown of the levee at its juncture with the ramp may be required to provide adequate turning radius. The grade of the ramp should be no steeper than 10 percent. Side slopes on the ramp should not be less than 1V on 3H to allow grass-cutting equipment to operate. The ramp should be surfaced with a suitable gravel or crushed stone. Consideration should be given to extending the gravel or crushed stone surfacing to the levee embankment to minimize erosion in the gutter. In general, private ramps should not be constructed unless they are essential and there is assurance that the ramps will be used. Unused ramps lead to maintenance neglect.

c. Both public and private ramps should be constructed only by adding material to the levee crown and slopes. The levee section should never be reduced to accommodate a ramp.

Section III
Levee Enlargements

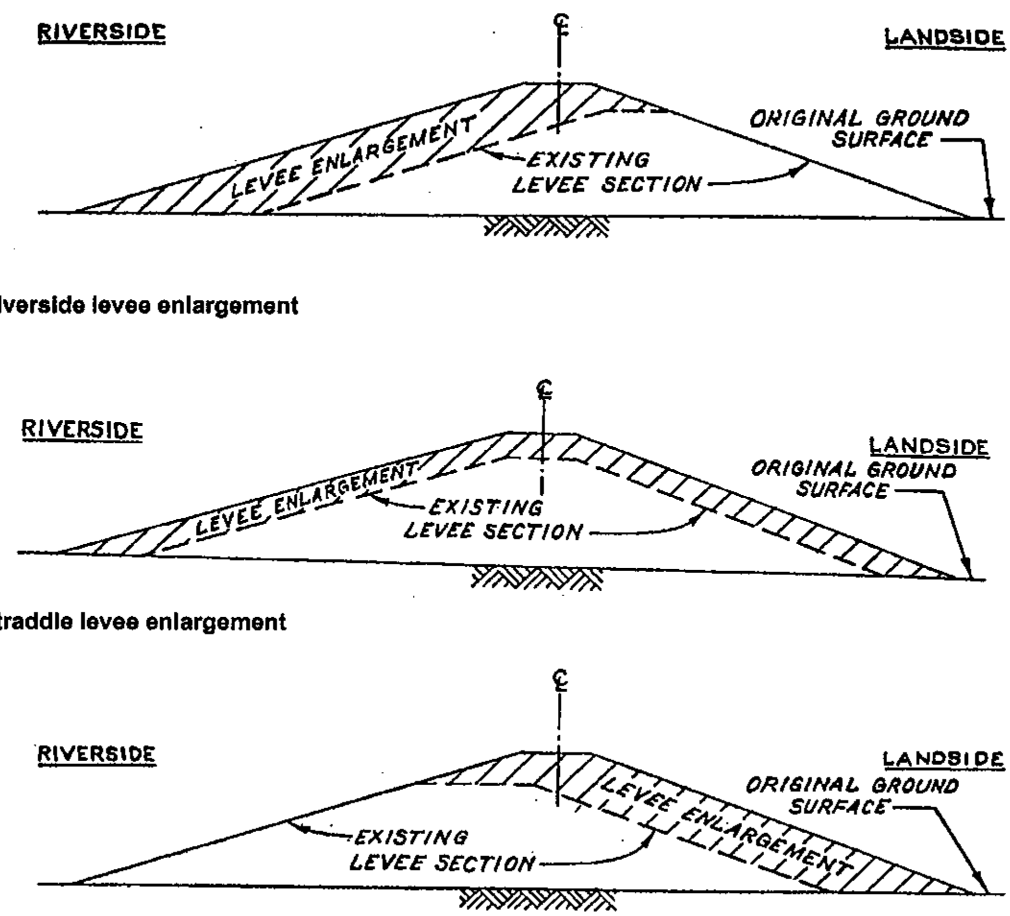
8-11. General

The term levee enlargement pertains to that addition to an existing levee which raises the grade. A higher levee grade may be required for several reasons after a levee has been constructed. Additional statistical information gathered from recent floodings or recent hurricanes may establish a higher project flood elevation on a river system or a higher elevation for protection from incoming tidal waves produced by hurricane forces in low-lying coastal areas. The most economical and practical plan that will provide additional protection is normally a levee enlargement. Levee enlargements are constructed either by adding additional earth fill or by constructing a flood-wall, "I"-type or "inverted T"-type, on the crown.

8-12. Earth-Levee Enlargement

a. The earth-levee enlargement is normally preferred when possible, since it is usually more economical. This type of enlargement is used on both agricultural and urban levees where borrow sites exist nearby and sufficient right-of-way is available to accommodate a wider levee section.

b. An earth-levee enlargement is accomplished by one of three different methods: riverside, straddle, or landside enlargement. A riverside enlargement is accomplished by increasing the levee section generally at the crown and on the riverside of the levee as shown in Figure 8-6a. A straddle enlargement is accomplished by increasing the levee section on the riverside, at the crown, and on the landside of the levee-as shown in Figure 8-6b. A landside enlargement is accomplished by increasing the levee section, generally at the crown and on the landside of the levee as shown in Figure 8-6c. There are advantages and disadvantages to each enlargement method that will have to be looked at for each project. The riverside enlargement would be more costly if the riverside slope has riprap protection and it could also be an encroachment for narrow floodways that would impact top of levee designs. Landside enlargements would require additional right-of-way and larger fill quantities for levees with flatter landside slopes. The strattle enlargement would require the whole levee system to be stripped with work being done on both sides of the levee.



a. **Riverside levee enlargement**

b. **Straddle levee enlargement**

c. **Landside levee enlargement**

Figure 8-6. Enlargements

c. The modified levee section should be checked for through seepage and underseepage as discussed in Chapter 5 and for foundation and embankment stability as discussed in Chapter 6. Sufficient soil borings should be taken to determine the in situ soil properties of the existing levee embankment for design purposes.

d. An earth-levee enlargement should be made integral with the existing levee. Every effort should be made such that the enlargement has at least the same degree of compaction as the existing levee on which it is constructed. Preparation of the interface along the existing levee surface and upon the foundation should be made to ensure good bond between the enlargement and the surfaces on which it rests. The foundation surface should be cleared, grubbed, and stripped as described in Chapter 6. The existing levee surface upon which the levee enlargement is placed should also be stripped of all low-growing vegetation and organic topsoil. The topsoil that is removed should be stockpiled for reuse as topsoil for the enlargement. Prior to constructing the enlargement, the stripped surfaces of the foundation and existing levee should be scarified before the first lifts of the enlargements are placed.

8-13. Floodwall-Levee Enlargement

a. A floodwall-levee enlargement is used, when additional right-of way is not available or is too expensive or if the foundation conditions will not permit an increase in the levee section. Economic justification of floodwall-levee enlargement cannot usually be attained except in urban areas. Two common types of floodwalls that are used to raise levee grades are the I wall and the inverted T wall.[1]

b. The I floodwall is a vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. For stability reasons, I floodwalls rarely exceed 2.13 m (7 ft) above the ground surface. One common method of constructing an I floodwall is by combining sheet pile with a concrete cap as shown in Figure 8-7. The lower part of the wall consists of a row of steel sheet pile that is driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel piling.

c. An inverted T floodwall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic pressures acting against the wall. A typical wall of this type is shown in Figure 8-8. The inverted T floodwall is used to make floodwall levee enlargements when walls higher than 2.13 m (7 ft) are required.

d. The floodwall should possess adequate stability to resist all forces which may act upon it. An I floodwall is considered stable if sufficient passive earth resistance can be developed for a given penetration of the wall into the levee to yield an ample factor of safety against overturning. The depth of penetration of the I wall should be such that adequate seepage control is provided. Normally the penetration depth of the I wall required for stability is sufficient to satisfy the seepage requirements. For the inverted T floodwall, the wall should have overall dimensions to satisfy the stability criteria and seepage control as presented in EM 1110-2-2502.

e. The existing levee section should be checked for through seepage and underseepage as discussed in Chapter 5 and for embankment and foundation stability as discussed in Chapter 6 under the additional hydrostatic forces expected. If unsafe seepage forces or inadequate embankment stability result from the higher heads, seepage control methods as described in Chapter 5 and methods of improving embankment stability as described in Chapter 6 may be used. However, some of these methods of controlling seepage and improving embankment stability may require additional right-of-way for construction which could eliminate the economic advantages of the floodwall in comparison with an earth levee enlargement. As in earth levee enlargements, a sufficient number of soil borings should be taken to determine the in situ soil properties of the existing levee embankment for design purposes.

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[1] Structural design of crest walls is given in ETL 1110-2-341.

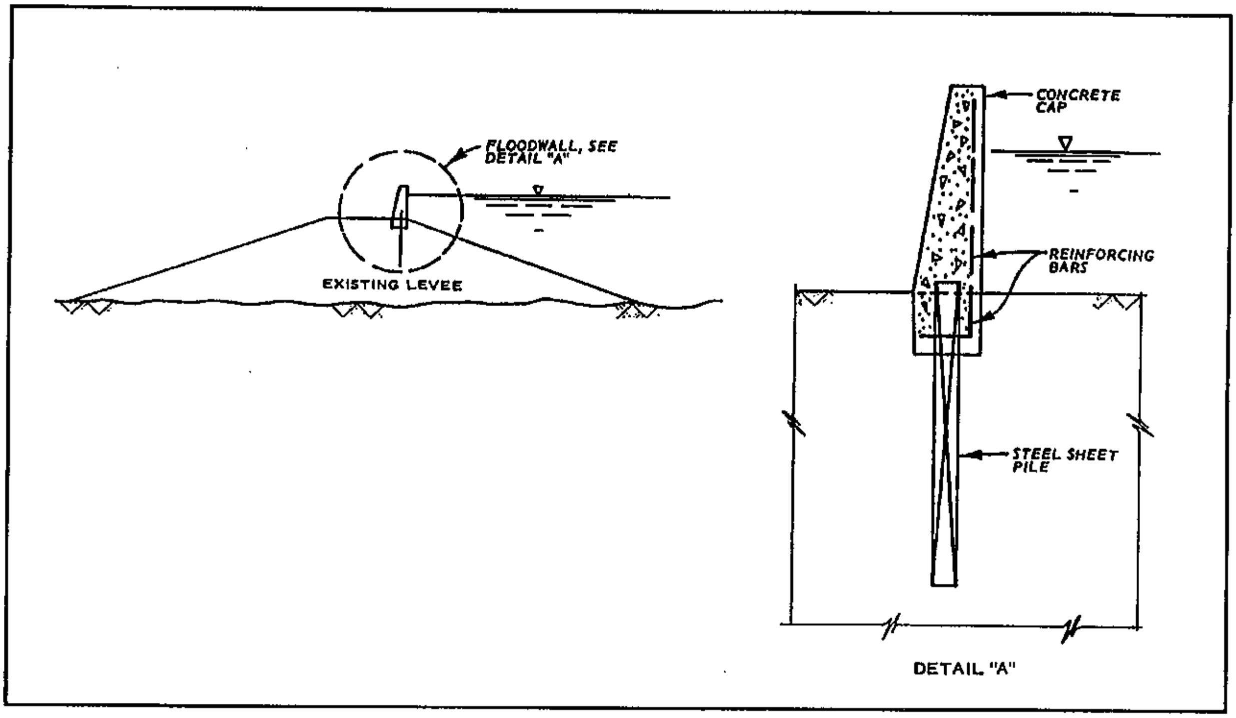

Figure 8-7. I-type floodwall-levee enlargement

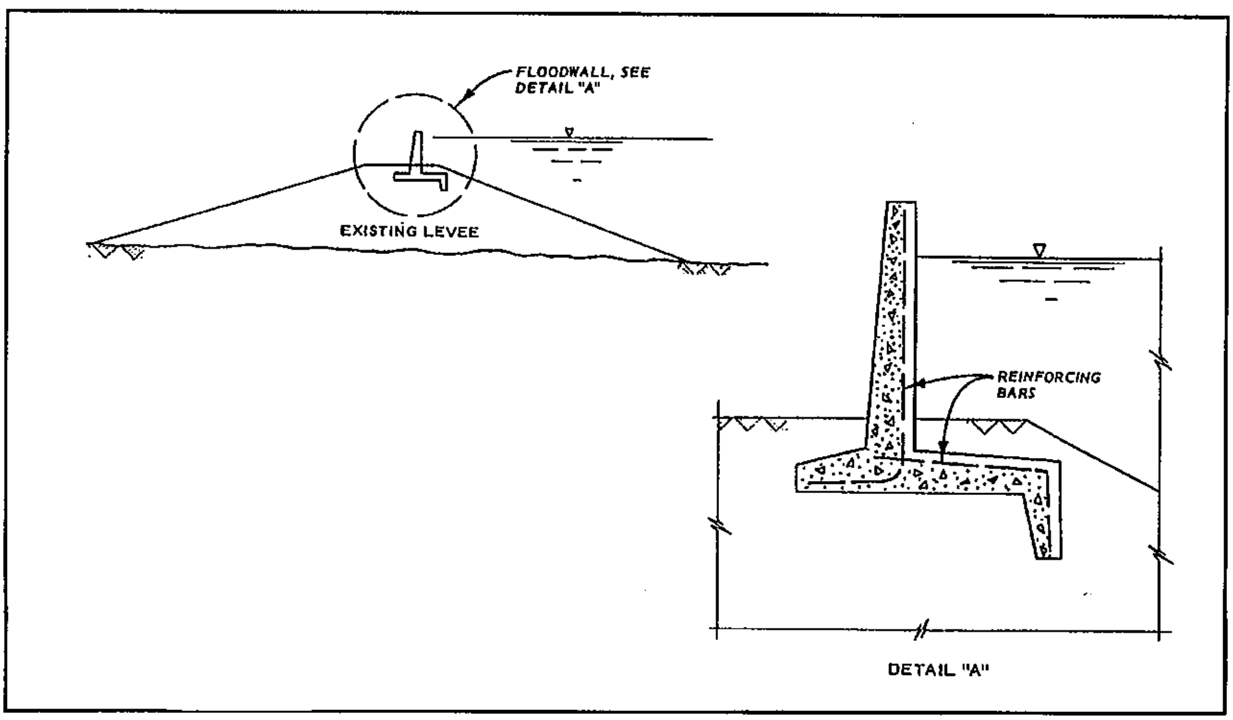

Figure 8-8. Inverted T-type floodwall-levee enlargement

Section IV
Junction with Concrete Closure Structures

8-14. General

In some areas, a flood protection system may be composed of levees, floodwalls, and drainage control structures (gated structures, pumping plants, etc.). In such a system, a closure must be made between the levee and the concrete structure to complete the flood protection. One closure situation occurs when the levee ties into a concrete floodwall or a cutoff wall. In this closure situation the wall itself is usually embedded in the levee embankment. In EM 1110-2-2502 a method of making a junction between a concrete floodwall and levee is discussed and illustrated. Another closure situation occurs when the levee ties into a drainage control structure by abutting directly against the structure as shown in Figure 8-9. In this situation the abutting end walls of the concrete structure should be battered 10V on 1H to ensure a firm contact with the fill.

8-15. Design Considerations

When joining a levee embankment with a concrete structure, items that should be considered in the design of the junction are differential settlement, compaction, and embankment slope protection.

a. Differential settlement. Differential settlement caused by unequal consolidation of the foundation soil at the junction between a relatively heavy levee embankment and a relatively light concrete closure structure can be serious if foundation conditions are poor and the juncture is improperly designed. Preloading has been used successfully to minimize differential settlements at these locations. In EM 1110-2-2502 a transitioning procedure for a junction between a levee embankment and a floodwall is presented that minimizes the effect of differential settlement.

b. Compaction. Thorough compaction of the levee embankment at the junction of the concrete structure and levee is essential. Good compaction decreases the permeability of the embankment material and ensures a firm contact with the structure. Heavy compaction equipment such as pneumatic or sheepsfoot rollers should be used where possible. In confined areas such as those immediately adjacent to concrete walls, compaction should be by hand tampers in thin loose lifts as described in EM 1110-2-1911.

c. Seepage. Seepage needs to be analyzed to determine the embedment length of the structure-levee junction. Zoning of the embankment materials needs to be maintained through the junction unless analysis indicates different zoning is required.

d. Slope protection. Slope protection should be considered for the levee embankment at all junctions of levees with concrete closure structures. Turbulence may result at the junction due to changes in the geometry between the levee and the structure. This turbulence will cause scouring of the levee embankment if slope protection is not provided. Slope protection for areas where scouring is anticipated is discussed in paragraph 7-6.

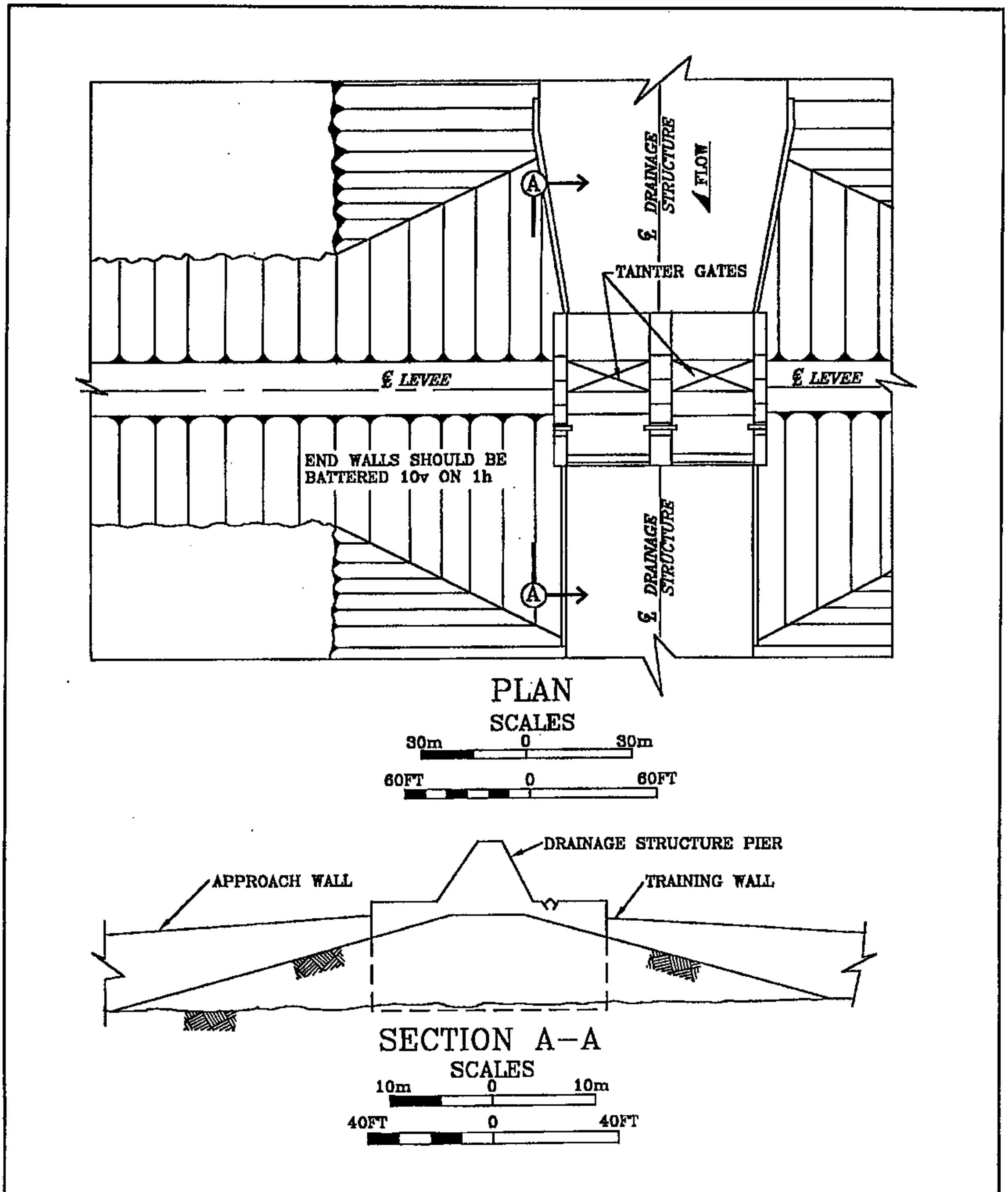

Figure 8-9. Junction of levee and drainage structure

Section V
Other Special Features

8-16. Construction of Ditches Landside of Levee

Sometimes requests are made to locate irrigation and/or drainage ditches in close proximity to the landside levee toe. Such ditches may lead to serious seepage and/or slope stability problems. The location and depth of proposed ditches should be established by seepage and stability analyses. This requires information on foundation soil conditions, river stages and geometry of the proposed ditch.

Drainage ditches should be located such that the exit gradient in the bottom of the ditch does not exceed 0.5 at the landside levee toe and does not exceed 0.8 at a distance 45.72 m (150 ft) landward of the landside levee toe and beyond. Between the landside levee toe and 45.72 m (150 ft) landward of the landside levee toe, the maximum allowable exit gradient in the bottom of the ditch should increase linearly from 0.5 to 0.8. The exit gradient should be computed assuming the water level in the ditch is at the bottom of the ditch.

8-17. Levee Vegetation Management

To protect or enhance esthetic values and natural resources, vegetation on a levee and its surrounding areas (trees, bushes and grasses) is an important part of design considerations. Vegetation can be incorporated in the project as long as it will not diminish the integrity and the functionality of the embankment system or impede ongoing operations, maintenance and floodfighting capability. A multidiscipline team including structural and geotechnical engineers, biologists and planners should evaluate the vegetation design or proposal. Coordination with local governments, states and Native American tribes may be needed during the design process. EM 1110-2-301 and ER 500-1-1 are two documents covering the vegetation policy applicable to both federal levees and non-federal levees under the PL-84-99 program.