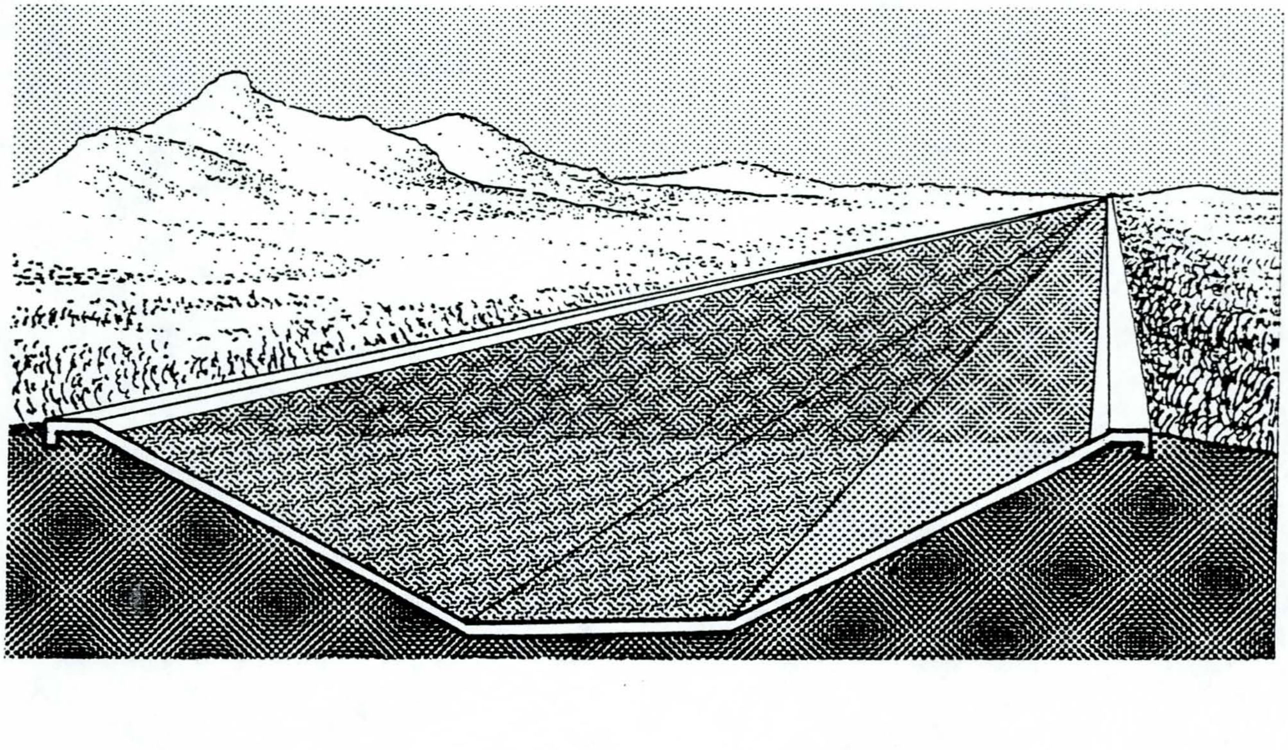
REPORT
ON
CONCRETE LINED DRAINAGE CHANNELS

FEBRUARY, 1989



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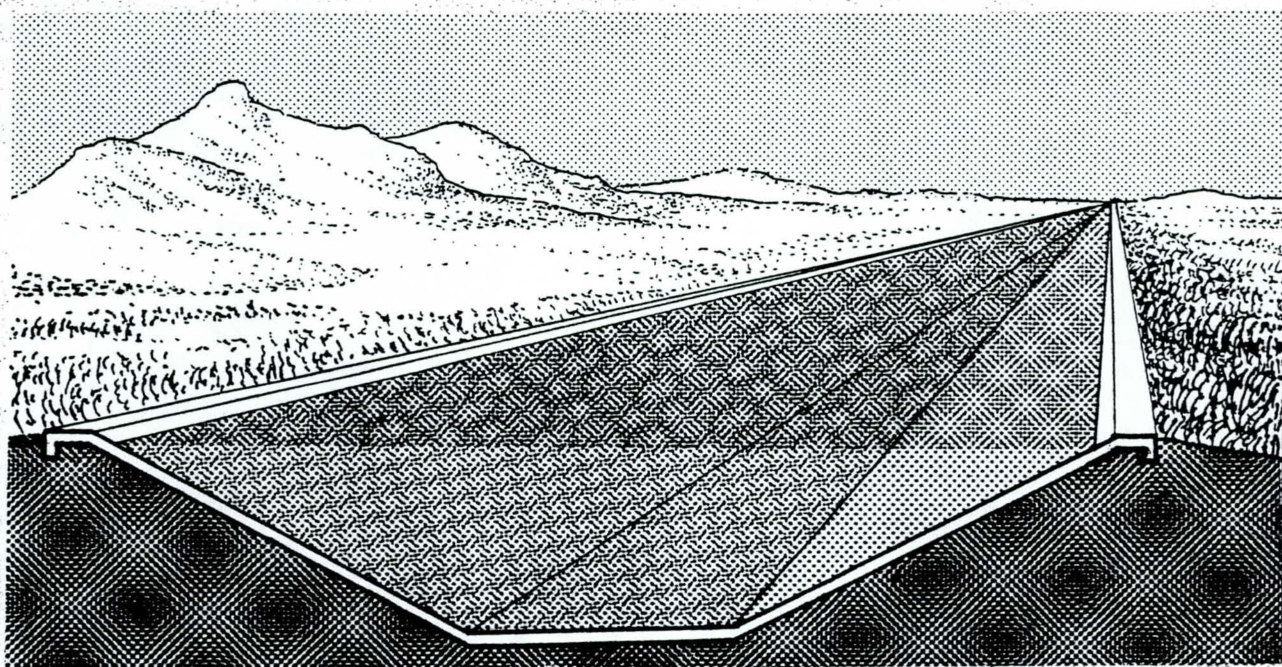


DMJM

DANIEL, MANN, JOHNSON, & MENDENHALL

DMJM EAST PAPAGO/HOHOKAM FREEWAY
MANAGEMENT CONSULTANT

EAST PAPAGO/HOHOKAM FREEWAY
CHANNEL LINING
DESIGN GUIDELINES

EXECUTIVE SUMMARY

ARIZONA DEPARTMENT OF TRANSPORTATION

URBAN HIGHWAYS
CHANNEL LINING DESIGN GUIDELINES

Introduction:

This publication is intended as an Executive Summary for channel lining designs. It was prepared to establish design criteria for concrete channel linings for the East Papago/ Hohokam Freeway project. This criteria can also be adopted for similar channel lining designs on other Arizona Department of Transportation (ADOT) projects. The study was conducted by gathering, reviewing and analyzing various published documents from Federal, State, and local agencies which are involved in the design and construction of concrete channel linings. These documents are included in a separate publication titled "Report on Concrete Lined Drainage Channels" dated February, 1989, which also contains this Executive Summary.

Design Criteria:

There are three distinct approaches to the design of channel lining reinforcement for canals and hydraulic drainage structures. Two of these approaches utilize expansion and/or contraction joints with the reinforcement varying from 0.0 to 0.5 percent depending on joint spacing. The third approach utilizes no joints with continuous reinforcing varying from 0.3 to 0.4 percent depending on climatic conditions. The design criteria of most agencies reviewed fall within one or more of these approaches and the resulting linings have performed satisfactorily for the most part.

The following items were considered in reaching the conclusion that a continuously reinforced lining without joints will provide the most cost-effective and serviceable channel lining:

- Review of lining performance relative to minimal cracking in continuously reinforced channels;
- Reported and observed maintenance problems of weeds growing in unsealed joints;
- High maintenance cost of replacing damaged joints;
- Moderate difference in construction cost between continuous versus discontinuous reinforcement;
- Potential local compressive buckling of lining due to open joints being filled with incompressible material;
- Potential infiltration of water through open expansion or contraction joints into moisture sensitive soils.

A 0.3 percent longitudinal reinforcement for a moderate climate was found reasonable for adoption to the Phoenix area, (it is performing well on the U.S. Army Corps of Engineers (Corps) Arizona Canal Diversion Channel). A minimum 0.2 percent transverse reinforcement for moderate climates is also considered reasonable for narrow to medium width channels up to 70 feet wide. For wider channels, it is considered realistic to increase the percentage based on the subgrade drag method to a maximum of 0.3 percent.

The criteria for the channel invert lining thickness follows two basic approaches and relates either to velocity or the presence of corrosive materials in the channel bed. Review of collected data indicates that the current general practice for establishing base slab thickness for channels without corrosive material follows closely the U.S. Soil Conservation Service (SCS) nomograph for thickness versus water velocity and it is concluded that bottom lining thickness should be based on this criteria. The minimum thickness, however, is dictated by two considerations--reinforcing clearance, and access for maintenance vehicles. Based on the support capacity of the stabilized moisture sensitive soils, a minimum thickness of 6 inches is required for maintenance vehicle access. A minimum 3-inch clearance for corrosion protection limits the minimum lining thickness to 6 inches for tied reinforcement and 5 inches for flat mesh.

Special consideration of thickness needs on a site specific basis will be required for unusual hydraulic and soil conditions. Energy dispators and extreme breaks in grade will require special design to ensure that lining thickness can resist potential negative pressures. Areas of collapsing or expansive soils require evaluation of thickness in conjunction with subgrade treatment to ensure a serviceable lining.

Slope paving thickness criteria (excluding SCS) generally approximates 80 percent of bottom thickness and is considered a reasonable approach. The limitations on minimum thickness for reinforcement and vehicle access applies to slope paving as well as bottom thickness. It was found that slope paving has even been placed vertically in transition areas, but it is the general consensus that a 1.5:1 maximum slope is more reasonable for maintenance of a quality product. In no circumstance should the slope exceed the soils angle of repose without being formed and designed as a retaining wall. In keeping with a continuously reinforced lining, it is concluded that transitions from trapezoidal channels to rectangular cross-sections should be accomplished without warped slope paving which the Corps is using on the ACDC (Figure 1).

A review of concrete qualities in the various standards indicates a wide range of values but with a general need for shrinkage and crack control. Concrete mix designs will be required which achieve a low drying shrinkage, while also maintaining strength requirements and constructibility.

A review of soil conditions in the Phoenix area indicates the area is well suited for construction of concrete lined channels. Most areas will require only a scarification and recompaction of surface soils without pressure relief. Minor areas of moisture sensitive collapsing or expansive soils will require partial or total removal and replacement with compacted fill based upon a site specific evaluation. Pressure relief and seepage barrier requirements in these moisture sensitive areas will also require special consideration.

The method of pressure relief on channel lining has generally been gravel pockets with weep holes. Problems with silting have been identified which create a maintenance requirement. Use of pressure relief flap valves in conjunction with geocomposite drainage strips is considered a better solution than weep holes. The need for pressure relief will require evaluation on a site specific basis not only for soil condition but for potential groundwater infiltration from heavy irrigation and special conditions such as parallel or crossing utilities.

The use of transverse cutoff walls has generally been eliminated from most design criteria (except at the start and end of a lining), and are not considered to be needed for elimination of seepage or progressive failure. A need does exist for transverse stiffening or stabilizing walls in continuously reinforced linings at movement sensitive structures and where unbalanced compressive forces may occur.

General details were reviewed to reduce construction and maintenance problems. Top of slope paving cutoff walls are a general practice and are needed to eliminate erosion and ground water seepage. A vertical wall set back from the top of slope sufficiently to allow machine trenching provides easier construction. A 2 percent cross slope to one side of the channel bottom (a minimum slope of 6 inches is recommended) provides a means to transport sediment during low flows. Access ramps should be located on the high side of the channel and slope downstream where possible to reduce hydraulic disturbance and sediment buildup. O-Gee control structures should be constructed with sufficient open area at the channel floor to allow flushing of sediment during low flows.

Recommendations:

We recommend the following "Design Guidelines for Concrete Lined Drainage Channels":

1. Channel lining shall be continuously reinforced without expansion or tooled joints except as follows. Construction joints shall be located at the end of a day's pour or when concrete placement stops for more than 45 minutes and between longitudinal paving strips. Longitudinal construction joints shall be located 1-foot up the side slope and in the bottom slab as dictated by channel width but not within the low flow section. Reinforcing steel shall be continuous through lining construction joints and through joints with box culverts and other hydraulic structures.

2. Reinforcing steel shall be Grade 60 or flat sheet welded wire fabric and have the following percentage ratios (p) of reinforcement to cross section area of concrete.

   Longitudinal Reinforcement: p = 0.30%

   Transverse Reinforcement:

Channel Width*	p
less than 70 feet	0.20%
70 to 90 feet	0.25%
more than 90 feet	0.30%

   *Total width including side slopes

   Reinforcing steel shall have a minimum 3-inch clearance to grade and a maximum size of #4 for 6 inch lining thickness.

3. Minimum lining thickness for trapezoidal channels shall be:

   Bottom Slab

Mean Water Velocity (fps)	Thickness (inches)
less than 10	5*
10 to 15	6
15 to 20	7
more than 20	8

   Side Slopes

Mean Water Velocity (fps)	Thickness (inches)
less than 15	5*
15 to 20	5 1/2
more than 20	6

   *Minimum slab thickness of 6 inches is required for use of tied reinforcement and in channels wide enough to accommodate maintenance vehicles.

Lining thickness and channel profile shall be investigated on a site specific basis where negative pressures might occur such as a change from a light to a steeper slope per Corps manual EM 1110-2-1602 and 1603.

Lining thickness and reinforcement shall be investigated on a site specific basis in conjunction with subsoil treatment where collapsing or expansive soils occur.

4. Side slopes on main channels should not exceed 1.5 horizontal to 1.0 vertical or the recommended maximum safe cut slope (Table 2) and preferably should not exceed 2.0 to 1.0.

   If side slopes which are steeper than the recommended safe cut slope are used for warped transitions, lining shall be designed as retaining walls for lateral earth pressures listed in Table 2.

5. Sealed vertical expansion joints shall be provided at bridge piers and abutments.

6. Transverse cutoff or stiffening walls which are rigidly attached to the paving shall be installed in the following locations:

   a. At the beginning and end of concrete lining unless terminating in a movement stable structure.

   b. Where new lining abuts an existing concrete lining that is not designed with continuous reinforcement. A transverse sealed expansion joint should be provided between new and existing linings.

   c. At the upstream or start of a transition section to widen the channel.

   d. At breaks in channel profile where the increase in slope exceeds 0.5 degree or 0.009 ft./ft.

   e. Immediately upstream and downstream of movement sensitive structures such as intersecting drainage channels. This shall be evaluated on a project specific basis.

7. Continuous 12-inch deep vertical cutoff walls with a top elevation 6-inches below natural grade shall be provided at the top of side slopes 2 foot back of the top of slope. The 2 foot horizontal section shall have a 2 percent slope toward the channel. Cutoff walls shall be increased to 24-inches deep where substantial flows occur.

8. Bottom lining shall have a cross slope to one side of 2 percent with a minimum of 6 inches of slope.

9. Access ramps shall be located upstream and downstream of box culverts and other hydraulic structures that will not allow vehicular access. Ramps should be located on the high side of the channel invert and slope in a downstream direction where possible.

10. Transitions from a trapezoidal cross section to a rectangular cross section should be made with a varying height vertical retaining wall (Figure 1) instead of warped side slopes. The retaining walls are to be designed for earth pressures listed in Table 2.

11. O-Gee control structures should be constructed with a 30 to 50 percent opening at the base slab for flushing.

12. Subgrade treatment shall be on a site specific basis in accordance with recommendations for the five typical subsurface profile cases in Table 1 and shall result in a minimum Modulus of Subgrade Reaction of 200 pci. Detailed discussions of subsurface profile cases and recommendations are found in Appendix A of the East Papago/Hohokam Freeway "Design Guidelines for Concrete Lined Drainage Channels."

13. Pressure relief of channel linings shall be accomplished with geotextile or geocomposite drainage strips and 4" diameter PVC weepholes through the lining in accordance with recommendations for the five subsurface profile cases in Table 1. Weepholes should be located 1-foot vertically above channel bottom and slope down 3" from back to face of lining. Plastic flap type relief valves should be considered if available and a workable detail can be developed.

    Project and site specific evaluation will be required based on subsurface investigations, potential future changes in ground water levels, where structural back fill occurs adjacent to channel, and at parallel or crossing utilities.

14. Concrete strengths, mix design and drying shrinkage evaluation shall be in accordance with recommendations in Table 3. Detailed recommendations for concrete design mix and shrinkage criteria are found in Appendix A of the East Papago/Hohokam Freeway "Design Guidelines for Concrete Lined Drainage Channels."

TABLE 1

RECOMMENDED SUBGRADE TREATMENT, DRAINAGE & PRESSURE RELIEF PROCEDURES FOR THE FIVE TYPICAL SUBSURFACE PROFILES IN THE GREATER PHOENIX AREA

Subsurface Profile Case	Description	Subgrade Treatment	Drainage & Pressure Relief
1	Clean sands or sands & gravels	• No special treatment required	
		• Scarification & recompaction of surface soils	• Pressure relief not required unless potential exists for groundwater to rise above canal bottom
2	Cemented desert alluvium	• No special treatment required	
		• Scarification & recompaction of surface soils	• Low risk of water accumulation: Pressure relief not required
			• High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise
			Geocomposite drainage strips with pressure relief weepholes
3	Moisture sensitive soils over poorly drained cemented desert alluvium	• <u>**Collapsing soils**</u>	• Low risk of water accumulation: Pressure relief not required
		• 4-feet thick: partial over-excavation, wetting, vibratory compaction & replacement with compacted fill	• High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise
		• Full removal & replacement with compacted fill	• Geocomposite drainage strips with pressure relief weepholes

TABLE 1 (Continued)

RECOMMENDED SUBGRADE TREATMENT, DRAINAGE & PRESSURE RELIEF PROCEDURES FOR THE FIVE TYPICAL SUBSURFACE PROFILES IN THE GREATER PHOENIX AREA

Subsurface Profile Case	Description	Subgrade Treatment	Drainage & Pressure Relief
3 (continued)		• **Expansive Soils** • Partial or total removal & replacement with compacted fill • Geomembrane underliner as seepage barrier	• Low risk of water accumulation: Pressure relief not required • High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise • Geocomposite drainage strips with pressure relief weepholes
4	Moisture sensitive soils over free draining granular stata	• **Collapsing soils** • 4-feet thick: partial over-excavation, wetting, vibratory compaction & replacement with compacted fill • Full removal & replacement with compacted fill • **Expansive soils** • Partial or total removal & replacement with compacted fill • Geomembrane underliner as seepage barrier	• Pressure relief not required unless potential exists for groundwater to rise above canal bottom

TABLE 1 (Continued)

RECOMMENDED SUBGRADE TREATMENT, DRAINAGE & PRESSURE RELIEF PROCEDURES FOR THE FIVE TYPICAL SUBSURFACE PROFILES IN THE GREATER PHOENIX AREA

Subsurface Profile Case	Description	Subgrade Treatment	Drainage & Pressure Relief
5	Expansive clays throughout profile	• Overexcavate & replace with nonexpansive compacted fill; depth of overexcavation as required to limit potential expansion to tolerable limits	• Low risk of water accumulation: Pressure relief not required
			• High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise
		• Geomembrane underliner as seepage barrier	• Geocomposite drainage strips with pressure relief weepholes

Note: Methodology for design of geocomposite drainage systems can be found in "Designing for Flow:", R.M. Koerner, Civil Engineering, Volume 56, No. 10, October 1986, and "Designing with Geosynthetics", R.M. Koerner, Prentice Hall International, New Jersey, 1986

TABLE 2

RECOMMENDED ENGINEERING DESIGN PARAMETERS FOR SUBSURFACE CONDITIONS 1 THROUGH 5

		*Slopes		Modules of Subgrade Reaction, pcl		Lateral Earth Pressures Against Retaining Walls: "Active" β, deg.				"At Rest" β, deg.			
Case	Subsurface Conditions	Cut	Fill	Dry	Wet	0	10	20	30	0	10	20	30
1	Clean sand or sand & gravel	2:1	2:1	600	600	30	31	37	56	50	52	61	93
2	Moderately to strongly cemented alluvial soils	1:1	1:1	750	600	30	31	37	56	50	52	61	93
3	Moisture sensitive (collapsing or expansive soils over cemented alluvium)	1:1	1:1	200	100	30	31	37	56	50	52	61	93
4	Moisture sensitive (collapsing or expansive) soils over granular free-draining soils	2:1	2:1	200	100	30	31	37	56	50	52	61	93
5	Medium to highly expansive clays throughout entire profile	1:1	1:1	600	400	30	31	37	56	50	52	61	93

*Notes:

1. Recommended slope ratios are horizontal to vertical. Slopes are maxiumum safe slopes. In most cases, slopes will be controlled by construction considerations and will be no steeper than 1.5:1.

2. Moduli of subgrade reaction for Cases 3, 4 and 5 for wet conditions are based on the moisture sensitive soils not being stabilized or replaced with structural fill. Values for dry conditions for these cases apply to stabilized moisture sensitive soils or structural fills.

3. "Active" case for lateral earth pressures applied to conditions in which the retaining wall is free to move at the top. The "at rest" case applies where walls are restrained from movement at the top. The angle β refers to the slope angle of the backfill from the horizontal.

TABLE 3

RECOMMENDED GUIDELINES FOR CONCRETE DESIGN MIX & EVALUATION OF DRYING SHRINKAGE

DESIGN MIX

- Design mix should meet the general specification requirements of ADOT 1006-3.

Strength

- Compressive strength should be 3,000 psi at 28 days.

Aggregates

- Aggregates should meet minimum requirements of ADOT Standard Specification 1006-3. Coarse aggregate should be size 57. Coarse aggregate should have a minimum of 75 percent crushed faces.

Mineral Filler

- Ninety (90) pounds of fly ash Class F (ASTM C618) shall be used as a mineral filler. Loss on ignition should be a maximum of 3.0 percent. Fly ash should not be considered as a replacement for cement. Fly ash should have an R factor less than 2.5. The R factor is defined as (C-5%)/F, where C is the calcium oxide content expressed as a percentage and F is the ferric oxide content expressed as a percentage. The R factor requirement may be waived if the contractor furnishes documented test results that the soil in contact with the Portland Cement concrete contains less than 0.10 percent water soluble sulfate, (as SO4) and/or the water in contact with the Portland Cement concrete contains less than 150 milligrams per liter sulfate (as SO4). The tests for sulfates should be performed in accordance with the requirements of California Department of Transportation Test Method No. 417. Calcium and ferric oxide content should be determined in accordance with the requirements of ASTM C311

Chemical Admixtures

- Should meet the requirements of ADOT 1006-2.04.

Water

- Should meet the requirements of ADOT 1006-2.02.

Cement

- Should be Portland Cement Type II, meeting the requirements of ASTM C150.

Slump

- Maximum 4 inches (AASHTO T119).

Air Content

- 5 plus or minus 2 percent by volume (AASHTO T-152).

TABLE 3 (Continued)

RECOMMENDED GUIDELINES FOR CONCRETE DESIGN MIX & EVALUATION OF DRYING SHRINKAGE

Curing

- Should meet the requirements of ADOT 1006-6 A.
- Subgrade shall be moistened and free of excess standing water prior to placement of concrete.

Hot Weather Concreting

- Should meet the requirements of ADOT 1006-5.02.

Minimum Cement Content

- Not applicable.

DRYING SHRINKAGE EVALUATION

Mortar Shrinkage Tests

- ASTM C157, "Length Change of Hardened Cement Mortar and Concrete," testing should be performed on the cement proposed for the project design concrete mix. If other than previously approved Type II cement is proposed, the shrinkage of the cement should be equal to or less than the value obtained in the control specimens made from previously approved cements which result in the lowest practicable shrinkage.

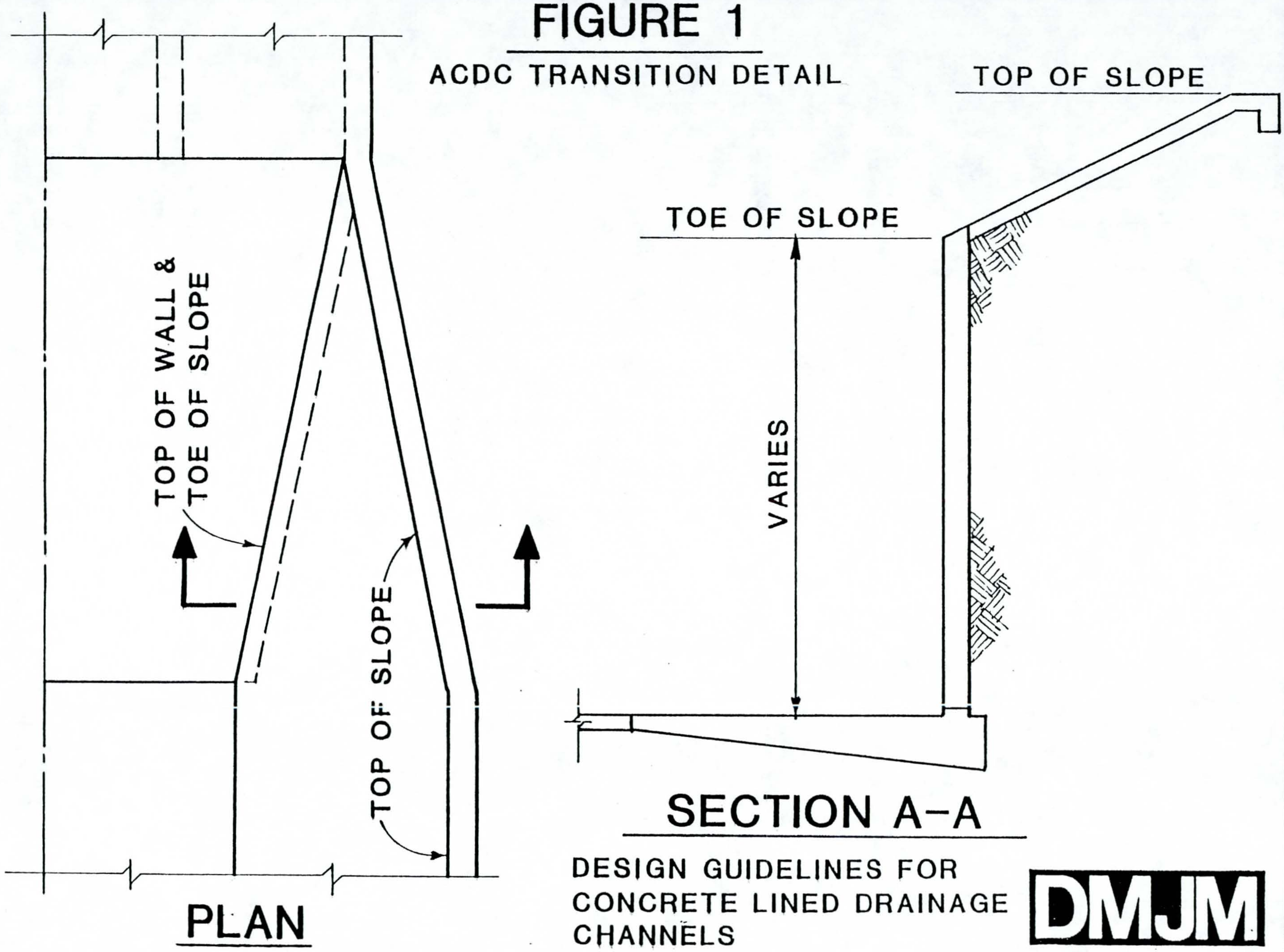
Field Shrinkage Tests

- Test panels should be prepared with the proposed concrete design mix for the purpose of evaluating drying shrinkage properties. Test panels should be made in accordance with the Kraai Method outlined in Concrete Construction, Volume 30, No. 9, September, 1985, 9 pp. 775-788. Test panels shall be 2 by 3 feet in plan dimension and 2 inches in thickness.
- The Control Test Panel should be made from an established reference mix design. Locally produced Salt River aggregate should be used. Minimum compressive strength should be 3,000 psi at 28 days. Fly ash, as a pozzolanic material, should be utilized as a mineral filler at a maximum of 90 pounds per cubic yard of concrete. A water reducing admixture should be used meeting the requirements of ADOT 1006-2.04.

  The project design mix acceptance should have an equal or reduced number and size of shrinkage cracking as compared to the Control Mix Test Panel.

FIGURE 1

ACDC TRANSITION DETAIL



DESIGN GUIDELINES FOR CONCRETE LINED DRAINAGE CHANNELS

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EAST PAPAGO / HOHOKAM FREEWAY DESIGN GUIDELINES FOR CONCRETE LINED DRAINAGE CHANNELS

REPORT

ARIZONA DEPARTMENT OF TRANSPORTATION

DESIGN GUIDELINES
FOR
CONCRETE LINED DRAINAGE CHANNELS

1. INTRODUCTION

By letter dated March 31, 1987, DMJM was requested to initiate a study to establish criteria for the design of concrete channel linings for the East Papago/Hohokam Freeway project, and as criteria that can be adopted for similar channel linings on other Arizona Department of Transportation (ADOT) projects. The study is to include the gathering, review and analysis of various published documents from Federal, State, and local agencies that are involved in the design and construction of concrete channel linings. In addition, telephone conferences are to be conducted with representatives of these agencies to verify current practices, design concepts, performance of linings and establish any major maintenance considerations.

Mr. Turan Ceran and Mr. James Tolle of DMJM attended a meeting on March 31, 1987 with Mr. C. Dennis Grigg, P.E., and Mr. Charles K. Eaton, P.E. of ADOT. Along with subsequent telephone conferences, this meeting established the basic design considerations for the channel lining study. Flows in the projected channels could vary from approximately 500 to 10,000 cfs but would be basically dry during the majority of the year. Channels with sufficient bottom width would be required to accommodate maintenance vehicles of the magnitude of H-10 to H-15 and have access ramps. Joint and cutoff wall spacing criteria should be reviewed as it relates to paving thickness and reinforcing requirements. The soil conditions based on a best and worst case consideration from existing reports should be evaluated for support and construction of proposed linings. Requirements for relief of hydrostatic pressures (weep holes, filter material, etc.) are to be considered based on varying soil conditions and presence of adverse external conditions such as utility trenches and runoff. Resolution of existing maintenance problems should be addressed and the final lining criteria should provide a 50-year life.

We have used the data provided by ADOT relative to concrete channel lining design criteria and/or standards as a base. We have obtained further data from the same and other agencies to update and supplement the original data provided in our meeting of March 31, 1987. The criteria collected and interviews conducted with agency representatives show diverse approaches to the design of concrete channel lining reinforcement and jointing. All approaches studied apparently perform to a reasonable degree of satisfaction with varying degrees of maintenance problems.

2. DATA REVIEW

Review of the collected standards or criteria for channel lining design indicate that there are three basic approaches to controlling or accommodating the inherent cracking that will occur due to shrinkage, flexural and temperature stresses in concrete linings.

One approach is the use of little or no reinforcement combined with close weakened plane or contraction joint spacing to maintain surface continuity so that cracks will occur at joints. Discontinuous reinforcing plus dowels at joints are sometimes provided to maintain surface alignment. The primary purpose of the light reinforcement is to hold tightly closed any cracks between joints so that aggregate interlock can function. The reinforcing does not prevent cracking nor does it increase significantly the load-carrying capacity of the slab. In this method, the joints are generally sealed with a sealant or water stop.

The second approach is to design the channels similar to continuously reinforced pavement without transverse or longitudinal joints except at the end of a day's run or when concreting has stopped for approximately 45 minutes. Even these joints are bonded and have continuous reinforcement. Longitudinal construction joints are utilized to facilitate paving strips. Expansion joints are provided only at bridge piers and abutments.

The third approach uses the subgrade drag method of analysis to establish the required area of reinforcing to control cracking between expansion joints. No weakened plane or contraction joints are used between expansion joints. Table 1 provides a summary of the major canal/channel lining standards or criteria that have been collected from specified agencies and reviewed. Other agencies were contacted but had insufficient criteria to be included in Table 1.

2.1 U.S. Bureau of Reclamation

The U.S. Bureau of Reclamation (USBR), although probably the most experienced in canal lining, does not provide a great amount of guidance for the design of erosion control channel thickness or reinforcement. The thickness recommended by the USBR[(1)] ranges from 3 1/2 to 4 1/2 inches at capacities of 500 to 5,000 cfs respectively. Velocities are generally less than 8 feet per second to avoid converting velocity head through a crack to pressure head and lifting the lining. This is at the lower end of the desired velocity range since per Mr. Raymond C. Jordan, P.E., Urban Freeway Drainage Engineer, ADOT, erosion control linings in the Phoenix area are generally required only when velocities exceed 6 to 7 feet per second. Also, these lining thickness would not be adequate for service by maintenance vehicles. In addition, the USBR criteria is for irrigation canal linings that remain filled with water and thereby have a different operating condition than drainage channels.

There are, however, a number of guidelines that can be drawn from the USBR[(1)] experience. Their experience indicates that the steepest satisfactory side slope for large lined channels from both construction and maintenance considerations is 1-1/2:1. Steeper slopes may be used on small laterals where the soil materials will remain stable. The lining requires a firm, dense foundation of existing or compacted earth with a smooth surface to receive the lining. The required degree of compaction varies with the soil, materials and thickness of backfill and must be given special consideration for each installation.

Compressive stress in channel linings from temperature increase is of little concern.[1] A slab which is fully restrained at both ends and subjected to a 100°F increase in temperature will develop only about 1,500 psi compressive stress. This is well below the average compressive strength of good concrete. Secondly, the expansion of concrete due even to complete saturation is never as great as the contraction that results from hardening and drying of the concrete shortly after placing.

Channel lining may be economically placed by a variety of methods. Large canals of considerable length can utilize longitudinally operated slip-forms supported on rails. Short lengths often do not justify slip-forms and may be lined by use of screeds operating transversely up each side slope.

Most USBR[2] canal structures are designed based on 3,000 psi concrete with 40,000 psi steel or 3,750 psi concrete and 60,000 yield steel to decrease cracks in the concrete. The concrete should be plastic enough for thorough consolidation and stiff enough to stay in place on the side slopes. Usually, a slightly oversanded mix is needed for machine-placed lining to give adequate workability. Close control of the workability and consistency of the concrete is important because a variation of 1 inch in slump can seriously interfere with the progress and quality of the work. An air-entraining agent is recommended but other admixtures such as set retarders have not proved fully satisfactory.

The proper curing of the channel lining is equally as important as the curing of any thin member. Moist curing is preferred but the use of accepted sealing compounds has been found satisfactory. To assure the lining does not dry out rapidly, the subgrade should be well moistened immediately prior to placing the concrete.

Don Anderson, Construction Engineer on the USBR Central Arizona Project (CAP) was contacted to establish if any major problems had developed on the project. The canal lining for the CAP project is 3-inch unreinforced slab with sealed control joints at 12 to 15-foot centers. No expansion joints were used. A slight amount of buckling of the thin 3-inch slab did occur during filling after the canal had sat empty for a long period. This occurred on the inside of curves and was attributed mostly to the temperature differential between the filled and unfilled portion of the lining. Only moderate amounts of buckling occurred. A slight amount of spalling did occur at joints before the canal was filled.

2.2 U.S. Army Corps of Engineers

The U.S Army Corps of Engineers (Corps) and the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) follow the second approach to lining design which is similar to the design of continuously reinforced concrete pavement. The Corps' criteria is contained in a 1978 Engineering Technical Letter ETL 1110-2-236 "Paved Concrete Flood Control Channels".[3] As stated, the Corps' experience has shown that the usual sealing compound will not give satisfactory performance when subjected to any sand scouring. Their experience has revealed that the more joints you have, the more maintenance is required. In general, the Corps' experience indicates that the paving should be designed as continuously reinforced, without transverse and longitudinal joints, except construction joints at the end of a day's run; between paving lanes; or anytime a pour stops more than 45 minutes. Vertical expansion joints are recommended where paving abuts another structure such as a box culvert or bridge pier. Expansion joints are normally provided with water stops to prevent overloading the subdrainage system.

An elaborate subdrainage system is recommended for linings where the water table is above the channel invert or where it is reasonable to anticipate a temporary water table rise due to seasonal variation or local ponding. Concrete strength is recommended to be a minimum of 3,000 psi with pavement thickness ranging from 4 inches on side slopes of small side channels to 10 inches for base slabs of main channels. Reinforcement for moderate and extreme temperature conditions is recommended to be 0.30 to 0.40 percent longitudinal and 0.20 to 0.27 percent transverse, respectively. A 2'-0" deep cutoff wall is recommended at the top of the slope paving for main channels where the paving stops below the level of the standard project flood. A one-foot wall is recommended otherwise.

Mr. Clifford Ford, Chief, Civil Design Section B, Los Angeles District, U.S. Army Corps of Engineers was contacted to discuss the criteria and guidelines in ETL 1110-2-236 and determine why the Corps had deviated from these guidelines on the design of the Arizona Canal Diversion Channel (ACDC).

Per Mr. Ford, the criteria in ETL 1110-2-236 was developed by the Los Angeles District where most of the Corps' experience has been gained. The L. A. District has a major problem with corrasion in their channels. Besides maintenance equipment, the channels carry fairly continuous low flow containing a bed load of abrasive materials. The low flow portion and even the main channel suffer substantial corrasion. The L.A. River channel, which had a 4-inch clearance to the reinforcement, eroded sufficiently to expose the rebar and required substantial repaving. The corrasion problems have resulted in the paving thickness of 10 and 8 inches for invert and slope paving respectively recommended in ETL 1110-2-236 for main channels.

The subdrain system recommended in ETL 1110-2-236 was also developed for the L.A. area where many of the channels were below the water table. Further discussion revealed that the Corps' had used expansion joints in their channels in the 1940's and 1950's but have had a lot of maintenance problems resulting in the need to replace the joints.

Relative to cut-off walls, it is the Corps' practice to use them only at the start and end of a lining. Mr. Ford indicated that the Corps had an incident occur during construction of a channel where a major flood occurred before the lining was completed. The flow got under the lining and damaged it for a distance of about 200 feet before dissipating. It was Mr. Ford's opinion that cutoff walls would be of little value since failures of this nature would proceed about 200 feet and not be progressive.

2.2.1 Arizona Canal Diversion Channel (ACDC)

When the ACDC was designed, it was realized that conditions in Arizona were substantially different than in L.A. The channel is dry most of the time and a study conducted by the Corps indicated that corrasion would be minor. It was, therefore, established that for a 100-year life, an 8-inch invert and 6-inch slope would be satisfactory for the 23,000 cfs design flow. The channel is constructed in sandy soils so no drainage system nor weep holes were provided. Expansion joints are provided only at the vertical face of bridge piers and filled with a joint sealant.

Discussions with Mr. Neil Erwin, Construction Engineer, on the ACDC project indicates that construction is proceeding on the channel's 6-inch side slopes with very acceptable results. There are minimal hairline cracks occurring at 10'± centers in the 6-inch slopes. One slight problem had developed during the earlier work on the slope paving. The concrete showed a slight bulge at each of the longitudinal rebars on the top layer. The exact reason for this was not determined but lowering the reinforcing 1/2 inch to provide about 2-3/4 inch clearance eliminated the problem.

Further discussions with Mr. Ford relative to paving thicknesses of 6-inch inverts and 5-inch slopes resulted in his opinion that these thicknesses of continuously reinforced lining would perform satisfactorily under the flow and corrasion conditions anticipated in the Phoenix area. He also indicated that, although ETL 1110-2-236 suggests a minimum slope paving thickness of 4 inches for small side channels, he was not aware of any case where the Corps had constructed any lining with less than 6-inch thickness.

Mr. Ford did indicate that smaller sized rebar or mesh would be needed in the 5-inch thickness to maintain clearance and eliminate the problem they had experienced on ACDC. Their experience also indicated that concrete with lower water/cement ratios provided better crack control and corrasion resistance qualities. His opinion was that from the Corps' experience, a continuously reinforced lining provides a more crack free and maintenance-free channel with a longer projected life. The Corps has had no major problems with the continuously reinforced linings without expansion joints.

2.3 Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA)

As previously noted, the AMAFCA design guide(4) follows the same basic approach as the Corps' recommendations in ETL 1110-2-236 but with a slightly reduced lining thickness requirement for the channel invert. A minimum of 7 inches increasing to 8 when velocities exceed 24 feet per second. These thicknesses seem more realistic when compared to the ACDC design. Reinforcement requirements, however, are more conservative than the Corps' with longitudinal reinforcement of deformed bars being recommended at 0.5 percent of the concrete area and a 0.25 percent transverse requirement. The 0.5 percent requirement is more in keeping with the current SCS practice but without joints. As with the Corps' criteria, expansion joints are omitted except at rigid structures and when tying to an existing unreinforced lining. Recommended concrete proportions are 5.5

sacks per c.y. with a minimum compressive strength of 3,000 psi, a 0.53 water/cement ratio and a maximum slump of 3-inches which is comparable to the ACDC project.

2.4 Subgrade Drag Method

The third basic design approach to channel lining reinforcement is the subgrade drag formula which prior to 1980 was the criteria used by the U.S. Soil Conservation Service (SCS). The subgrade drag method is also recommended by ACI(5) for slabs on grade with a slight modification. In the ACI publication SCM-5(83), a value of 0.75 of the yield strength is recommended because the consequence of a reinforcement failure is much less important than in normal reinforced concrete structural work.

The subgrade drag theory is explained in Appendix A of SCS's 1966 Engineering Design Standards Far West States(6) as follows:

> In trapezoidal channels restraint to slab movement is caused by location of cutoff walls, certain types of joints, and the weight of concrete.
>
> Rectangular channels are restrained by lateral soil pressure on the sidewalls, the weight of soil on the heel of the wall, the weight of sidewalls as transmitted to the invert of the channel as well as the weight of the invert, location of cutoff walls, and certain types of joints.
>
> Short slabs of concrete need no reinforcement since plain concrete is able to develop some tensile strength to resist minor dimensional changes. Longer slabs will require reinforcement in proportion to the slab length because the restraint to movement as a result of friction

between slab and subgrade increases with slab length and will exceed the tensile strength of the concrete.

The basis of the design criteria is the principle that "the amount of reinforcing required to reduce the size and spacing of concrete cracks is a function of the restraint imposed on the slab movement due to temperature variation."

Development of this principle by means of simple static analysis results in the subgrade drag equation and in its common form (with modified definitions) is:

$$A_s = \frac{CLW}{2fs}$$

where

A_s = effective cross-sectional area of steel in square inches per foot length (or width) of section.

C = Coefficient of subgrade friction.

L = distance in feet between unrestrained ends of slab. (If the ends are restrained, an equivalent length should be used.)

W = weight of the slab in pounds per square foot. (An equivalent weight of slab should be used for rectangular channels.)

f_s = allowable unit tensile stress in the reinforcing steel in pounds per square inch.

This equation with appropriate values for constants C and f_s has been plotted in graph form in Figure A-2.

The graph is included in Appendix B of this report.

2.4.1 U.S. Soil Conservation Service

The U.S. Soil Conservation Service (SCS) design criteria is found in their "Far West States Design Standards"[6] and "Technical Release Number

67."[7] (Appendix B). Recommended paving thickness is determined based on a nomograph (Fig. 1.7 Appendix A, Far West States Design Standards) and is related to water velocity. Comparing the California Department of Transportation's (Caltrans) recommended thickness vs. velocity criteria with SCS indicates reasonable agreement between the recommendations. Even the AMAFCA recommendation of an 8-inch thickness for velocities in excess of 25 feet per second is not unrealistic per the nomograph.

The SCS design criteria for minimum reinforcement is specified in their 1980 "Technical Release Number 67." Reinforcement for joint spacing less than 30 feet is 0.3 percent with a 0.5 percent requirement for joints greater than 30 feet. Mr. Paul Monville, Acting Head, Design Unit, SCS West National Technical Center, Portland, Oregon. was contacted to discuss current design practice. Mr. Monville indicated that the subgrade drag method is currently used in conjunction with the minimum requirements in the design of reinforcement. The subgrade drag formula is used to establish the maximum joint spacing for 0.5 percent reinforcement or to establish the amount of additional reinforcement required for a greater desired joint spacing.

With a 0.5 percent reinforcement requirement and inclusion of expansion joints into the design, the SCS approach becomes more conservative than the Corps. SCS also recommends cutoff walls at rigid structures and at a maximum 1,000' spacing to reduce the potential of progressive failure.

2.4.2 SCS Roosevelt Water Conservation District Channel

The Phoenix office of SCS was contacted to discuss their current design practices. It was found that on the Roosevelt Water Conservation District (RWCD) channel that a 6-inch trapizodal lining with #5 at 10" was being used for a flow of 5,600 cfs. Expansion joints of 3/4-inch width are spaced at 100 ft. centers and filled with sealant. Construction joints between paving strips were used in the longitudinal direction with the reinforcement continuous through the joint. Contrary to SCS recommendations, no transverse cutoffs were used. Based on the slab thickness and area of steel provided (0.05 percent), the expansion joints could have been spaced at about 180-foot centers by the "Subgrade Drag" design approach which makes this a very conservative design. By the Corps' standard, the reinforcing is more than adequate to be continuous without expansion joints.

2.4.3 Power Line Floodway Channel

Mr. Ralph Arrington, Arizona State Engineer, U.S. Soil Conservation Service, was later contacted relative to reported buckling failures that occurred on the Power Line Floodway near Phoenix. He explained that about 5 years after the channel was constructed, there was a 20-day period of 110° temperature that caused the extreme expansion and buckling.

The channel has a 5-inch thick slab reinforced with 6x6-6/6 wire mesh. Expansion joints are located at 2-foot deep cutoff walls at 325-foot centers. Dowels were not provided through the 1/2-inch expansion joints. Tooled contraction joints are spaced at 50 foot centers between cutoffs with the reinforcing continuous.

The intermediate contraction joints cracked and opened due to shrinkage and contraction, subsequently being filled by the fine desert sands. Even some of the expansion joints were infiltrated with sands. Expansion during the 20 days of 110° temperature squeezed the joint filler out of the expansion joints. The top half of the slabs were spalled off at the joints and the lining lifted and buckled at about 10 percent of the expansion joints. Mr. Arrington was uncertain but thought some buckling had occurred between expansion joints. Their plan for repair is to saw cut expansion joints at each tooled joint.

2.5 California Department of Transportation

The California Department of Transportation (Caltrans) Highway Design Manual[8] (January 1987) provides a limited degree of guidance for design of channel linings. As shown in Table 1, they recommend various lining thicknesses and reinforcements based on velocity. As noted above, the thicknesses agree reasonably well with SCS. The manual, however, provides no criteria for joint spacing, joint type nor cutoff wall spacing except for slope paving protection. In an attempt to establish current practice several District offices were contacted with little success. Mr. Keith Herman, P.E., Flood Control Coordinator, in their Los Angeles office finally provided some information. He indicated that the standard specifications called for sealed expansion joints at 20' spacing but it was his opinion, however, that joints were not being spaced quite that close. Cutoff walls were definitely not being used and no specific problems with the use of the recommended thicknesses had been brought to his attention. Applying the "Subgrade Drag" method for reinforcement to Caltrans standards, the expansion joint spacing for the Caltrans linings should be in the range of 40 feet. This would indicate the recommended reinforcing is satisfactory based on the reported joint spacing and that the Caltrans criteria is somewhat similar to the criteria used by SCS prior to 1980.

The report "Bank and Shore Protection in California Highway Practice" (1970 reprint)[9] is the results of a study conducted in 1949 for the California State Highway Engineer. Although not directly relating to channel linings, the report does offer some guidance on design of erosion protection for channel slopes. There is no specific criteria relating lining thickness to flow but it does recommend 3 or 4 inch for light duty and 4 or 6 inch for heavy duty slope paving. The expansion joint spacing of 20 to 30 feet on heavy duty and 60'± for the light duty paving generally follows the SCS approach to channel design. Reinforcement is designed by the subgrade drag method which is in agreement with the pre-1980 SCS criteria. Cutoff walls to guard against progressive failure are recommended at ends and 20 to 30 foot intervals. In general, the criteria appears to parallel that of the pre-1980 SCS criteria and has not incorporated the more recent 0.3 and 0.5 percent SCS minimum reinforcement requirements.

2.6 Los Angeles Flood Control District

The City of Los Angeles Flood Control District (LAFCD) was contacted based on reports they had a manual for the design of trapizodial channels. Mr. Dan Short, Supervising Civil Engineer III, informed us that they did not have any current published standards but had developed some preliminary guidelines. Their practice has been to use a minimum 8-inch thickness for inverts and 6-inches for side slopes. Expansion joints are spaced at 50-foot centers with a reinforcement percentage of 0.18 percent. They have had problems with high velocity flows sucking the filler material out of expansion joints. Subsequent filling of joints with sediment and temperature expansion resulted in spalling at the joints. Joints have since been cleaned and filled with an elastomeric sealant but continue to be a major maintenance problem. Mr. Short felt that the expansion joints were the major problem they have with channels. Cutoff walls are not being used.

They have also had problems with weep holes plugging and are currently using plastic back-flow flap valves instead of weep holes to relieve hydrostatic pressure behind linings. They use higher strength concrete (5000 psi) in the invert when corrasive material is anticipated. Comparing minimum paving thickness criteria to the Corps indicates the LAFCD agrees on the need for thicker slabs in the L.A. area due to corrasion problems. The reinforcing and expansion joint requirements, however, are more in agreement with the "Subgrade Drag" approach to lining design. Applying "Subgrade Drag" criteria to the 8-inch thickness with 0.18 percent steel results in the requirement of a 60-foot expansion joint spacing. This compares well to the 50-foot spacing being used. The L.A. County Flood Control District appears to be using a mix of Corps and pre-1980 SCS criteria.

2.7 San Diego Regional Standards

The San Diego Region (1983) criteria included in Table 1 was obtained from two standard drawings titled, "Major and Minor Drainage Channels". Contact with the San Diego Regional Standards Committee office revealed that the two drawings constitute their full criteria and there is no documented design approach. Slab thickness recommendations are in close agreement with both Caltrans and SCS although there is no correlation between slab thickness and flow. The reinforcement and joint spacing standards, including filling of joints with filler, are more consistent with USBR criteria. The moderate amount of reinforcing combined with close joint spacing should maintain paving continuity by aggregate interlock and concentrate most of the cracking at the joints. The amount of joint filler required with these standards, no doubt, requires substantial maintenance.

2.8 Ventura County Flood Control District

A section on the design of Reinforced Concrete Trapezodial Channels from the Ventura, California, County Flood Control District's Design Manual, included in Appendix B, was obtained from Keith Herman of Caltrans. The thickness standards of 6 and 7 inches for velocities of below and above 25 fps are in line with other agencies but their reinforcement standards are far more conservative than most when you consider they require rebars be discontinuous at the 20-foot spaced contraction joints. Even with the "Subgrade Drag" approach, #4 @ 12" would allow joint spacing of 100 feet and the Corps standards would eliminate joints. They also introduce another design criteria for their transverse reinforcement that has not appeared in any other standard. They require the side slope walls be designed for 50 percent of the moment resulting from an equivalent fluid pressure of not less than 40 psf plus any surcharge. This could, in deep channels, require a very substantial increase in both slab thickness and reinforcement.

2.9 Salt River Project

Mr. Tim Stanton of the Salt River Project was also contacted to determine their current practices on channel linings. Since their canals are for irrigation, they basically follow the USBR approach. Linings consist of 1-1/2" slip formed on small laterals with 2" shotcrete or 4" concrete, all unreinforced, on main canals. They have had some breakup of the linings but most have performed satisfactorily. The canals are full of water most of the time so are not truly representative of the conditions expected in the erosion control channels.

2.9.1 DeLeuw Cather "Drainage Channel Lining Reinforcement" Report

In addition to the material already discussed, we have reviewed the material contained in Appendix A and B of the DeLeuw Cather "Drainage Channel Lining Reinforcement" report that was transmitted to DMJM on April 20, 1987. Most of the material in their Appendix A has already been discussed along with additional criteria from the Corps, SCS, and other agencies. The standards for Maricopa County, Pima County, and the City of Tucson presented in the report are much like the USBR and provide little additional guidance.

The information in the report under the Section "Los Angeles County Flood Control District Criteria" had not been obtained. Sections P, Q, and R from the LACFCD structural Design Manual on the Design of Open Trapezoidal Channels, however, deals only with subdrainage systems and does not provide any standards on lining thickness or reinforcement. It is our understanding from Mr. Dan Short of the District, as previously discussed, that there are no published standards. It is also our understanding that the "Checkers Guide to Trapezoidal Channels" which is marked "Not Necessarily Policy" is only under review by the District. The slab thickness used in the design example of 10 and 8 inches for invert and slope pavement are greater than the 8 and 6-inch thicknesses reported to us. The required reinforcing percentage is basically the same and the expansion joint spacing is only 10 feet greater than the information we obtained from the District. Designing the slabs to span across a three foot void would seem a little severe since providing a well-compacted, stable subbase for a channel lining is generally considered a better approach.

3. EXISTING CHANNEL INSPECTION

Inspection of channel linings was conducted on April 16, 1987 by Mr. Turan Ceran and Mr. James Tolle of DMJM accompanied by Mr. Dan Lance and Mr. Larry Harris of ADOT.

3.1 Superstition Freeway Channel

The Superstition Freeway channel from Val Vista Drive to Higley Road was checked in a number of locations. The channel is relatively small with a 2-foot bottom width and no access for maintenance. The typical section has 6x6 - W1.4xW1.4 mesh in a 4-inch slab with 2:1 side slopes. 1-1/2 inch deep tooled weakened plane (contraction) joints are located at 20' centers longitudinal and transverse. No weep holes were provided. At channel transitions, the slab is thickened to 6 inches and #4 @ 12" transverse reinforcing added in the lower 4 feet of the slope paving in the 15-foot length adjacent to the culvert.

The lining is in good condition with the major cracking having occurred in the contraction joints. Occasional hairline cracks were observed in the lining between joints but were minor. One area of slope paving had suffered substantial cracking during construction when the supporting soils were eroded allowing the slab to defect downward. This had been repaired by crack injection and grouting of the voids behind the slab. The repaired lining appears to be performing quite well.

One major maintenance problem does exist on the Superstition channel. The top edge of the lining on the south side was not carried to the existing ground line for most of the length. Surface flow has eroded behind the top cutoff wall in many locations and required paving with asphalt. Since the channel is too narrow for vehicle access, it is difficult to clean and there is substantial silts and sand in the invert.

3.2 Ehrenberg - Phoenix Highway Channel

The Ehrenberg - Phoenix Highway channel lining was also inspected in a number of locations. The channel width varies but is wide enough and has access ramps for maintenance vehicles. The typical section has 6x6 - W10xW10 mesh in a 6-inch slab. Most of the side slopes are 2:1 but there are sections between 67th and 99th Avenues that have 1-1/2:1 and 1:1 slopes. The invert slab slopes to one side to provide a low flow channel. Weep holes are provided on each side slope. One and one-half inch tooled joints in both directions are provided at 20 ft. centers. One-half inch wide asphalt cement sealed construction joints were used at the slope and base intersection from Aqua Fria to 95th. A center longitudinal joint with sleeper slab and 3/4 x 3/4 sealant was used between 95th and 49th whereas three alternate joint configurations were offered between 79th and 43rd. All sections were constructed with the reinforcement continuous through the joints.

Performance of the lining is good with few cracks noted between the tooled joints. Cracks do occur in the joints as expected and Mr. Lance reported that some joints have opened as much as 3/8". Joint sealant is missing in many areas and Mr. Lance indicated it is a major maintenance problem. One short section was noted where a slight uplifting and spalling of one side of the base slab had occurred along the center joint. Mr. Lance also mentioned that during construction on one channel they were unable to keep asphaltic filler in the joints due to high temperatures.

The top edge of the lining extends to ground line so there is little erosion problem. Another maintenance problem is the large weeds that grow in the sediment in cracks at joints, in weep holes, and where sediment build-up has occurred due to changes in channel flow characteristics. Three particular instances were noted. An O-Gee wier which had heavy buildup on the upstream side. A short section had been removed to allow sediment to move through during lower flows. A section of channel where it widened to pass under an irrigation canal was heavily silted and locations where access ramps sloped down in the upstream direction caused silt buildup. Weep holes plug up in areas of silt buildup and are difficult to maintain.

Mr. Lance also noted that problems have occurred with hydrostatic pressure behind linings where utility trenches pass below channels. They have also had hydrostatic problems in non-weep hole channels where erosion has allowed surface flow behind the linings. It seemed that, everything else considered, maintenance of expansion joints was the major problem and their elimination preferred.

3.3 Outer Loop Highway Channel

The channel lining currently being constructed on the Outer Loop was also visited. Some lining had been constructed using the original design with 6x6 - W1.4xW1.4 mesh in the 6-inch lining. It was not inspected closely but appeared in good condition. Placement of the revised reinforcement of #4 @ 12" was proceeding on the side slopes but no concrete was being placed.

Drawings on the project were provided by ADOT which show some different practices than were used on the existing linings. Invert cross slope, top of lining detail, weepholes and 1-1/2" tooled joints were basically the same as the Ehrenberg-Phoenix Highway. No longitudinal joints were used other than tooled and construction joints but 3'-6" deep transverse cutoff walls had been added at 500 ft. centers. The lining, however, was not bonded to the top of the cutoff wall. Transition sections appear to be designed for lateral earth pressure since they have varying thickness with increased reinforcement.

3.3.1 Outer Loop General Contractor

Mr. Mike Murphy, Sundt Construction Co. and Paul Curry, Rapel Steel, the contractor and his reinforcing subconsultant on the Outer Loop Channel were contacted later to review construction problems and establish why #4 @12" reinforcing was favored over 6x6-W10xW10 flat sheets. Mr. Murphy indicated they were using a Bid-Well Trimmer to place a 2" slump concrete on the slopes and having good results. He was having no problems with the details and is constructing the slopes plus 2 feet of the invert before placing the base. He indicated that his rebar supplier had elected to use tied steel in lieu of flat mesh because of cost but felt they may have been influenced by a shortage of work for their crews and lack of experience in using flat sheets.

Discussions with Mr. Curry revealed there were some problems with the use of flat sheets. Bending of the sheets is done in California and shipment becomes a problem. There is a problem of fitting prebent sheets at the top since it varies with the existing grade. Even the bottom is some problem since excavation is not uniform. For mesh to be competitive a constant uniform cross section would be required. Even with that, he felt the need for a crane to handle the heavy sheets would make the mesh more expensive. Further checking with other suppliers confirmed that mesh is generally not competitive with rebar when mesh exceeds 6x6-W4xW4.

4. MAINTENANCE VEHICLE REQUIREMENTS

Based on discussions with Mr. Dan Lance, the H-10 to H-15 truck loading for maintenance vehicles may be a little high since most cleanup is done with a front end loader. Since no precise load is known, the H-10 to H-15 range load has been used in our review.

Due to the low vehicle volume expected in the channels, the Portland Cement Association (Appendix B) publication "Design of Concrete Pavement for City Streets"(10) was used as a guide to establish the required lining thickness. A "Light Residential Street" is defined as one where "Traffic volumes are low, less than 200 vehicles per day with 1 percent to 2 percent heavy commercial traffic. Trucks using these streets will have a maximum tandem axle load of 36 kips and a 20-kip maximum single-axle load." The 16 to 24-kip single axle load of the H-10 to 15 truck with a volume less than 2 to 4 vehicles per day meets this criteria.

Per DCPCS(10) "Because of its rigidity, concrete pavement has remarkable beam strength and load carrying capacity, thus the pressures beneath the concrete pavement are very low and distributed over relatively large areas. The ability of concrete to distribute heavy loads makes it unnecessary to build up subgrade strength with thick layers of crushed stone or gravel. Therefore, economical concrete pavements that will give good performance can be built on most in-place soils." As discussed in the Geotechnical Report, Appendix A, some site specific considerations will be required in the areas with collapsible or expansive soils.

The supporting power of the subgrade is expressed as values of k, the subgrade reaction. As discussed in the Geotechnical Report, the soils in the study area covered by the furnished reports fall in five categories with subgrade reactions varying from a low of 200, when the moisture-sensitive soils are stabilized to a high of 360.

Since the critical stresses in concrete pavement are flexural rather than compressive, the flexural strength of concrete (expressed as Modulus of

Rupture, MR) is used in concrete pavement design. The expression for the modulus of rupture is:

$$MR=K\sqrt{f'c}$$

where K can range from between 7 and 13 for stone concrete. Using the conservative ACI valve for K of 7.5 and 4,000 psi concrete MR = 475.

Using Chart 1 for residential streets with 35-year life from DCPCS[10], approximately four 2-way heavy vehicles per day, K = 200 and MR = 475, it is found that the required design thickness is approximately 6.1 inches. With the subgrade modifications discussed in the Geotechnical Report a 6-inch lining will be adequate for maintenance vehicle traffic even in most areas with moisture-sensitive soils. In some isolated site specific locations the linings may require special designs as discussed in the Geotechnical Report.

5. HYDRAULICS

5.1 Arizona Department of Transportation

Mr. Raymond C. Jordan, Urban Freeway Drainage Engineer, ADOT was contacted to determine current policy for the hydraulic design of channel linings within the general Phoenix area. We were informed that the present design standard is that drainage channels should accommodate with normal freeboard the runoff from the 50-year rainfall event. This will generally handle the standard 100-year return period flow with minimal freeboard and no significant adverse flooding. Most agencies' policies, Corps, SCS, etc., require a 100-year return period with freeboard.

Mr. Jordan indicated that linings in the Phoenix area are generally required when velocities exceed 6 to 7 fps to prevent erosion damage. Some channels have reached velocities of 16 to 17 fps. ADOT's current practice on the Outer Loop Highway has generally been to keep the Froude number at 0.8 or less. He also noted that they were sloping the channel inverts to one side to create a low flow channel to reduce silting problems.

5.2 Freeboard

5.2.1 U.S. Bureau of Reclamation

Review of the policy of various agencies indicates similar, but slightly different standards. The USBR varies its recommended freeboard based on canal capacity from a minimum of 6 inches at 30 cfs up to 3 feet for 20,000 cfs.

5.2.2 U.S. Soil Conservation Service

The SCS in their Design of Open Channels, Technical Release No. 25[11], recommends that freeboard for trapezodial channels at subcritical flow should be equal to or greater than 20 percent of the depth at design discharge but not less than one foot. They also recommend that curve radii be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or 10 percent of water surface width, whichever is less. Their formula for superelevation is:

$$S = \frac{V^2(b+zd)}{2(gR-2zV^2)}$$

where the terms are:

b = bottom width of channel (feet)
d = depth of flow (feet)
g = acceleration of gravity
R = radius of curve (feet)
s = superelevation (feet)
V = average velocity (fps)
z = cotangent of side slope angle

5.2.3 Albuquerque Metropolitan Area Arroyo Control Authority

The Albuquerque (AMAFCA) design guide is somewhat more conservative in its freeboard requirements using the formula:

$$\text{Vertical Freeboard (feet)} = 2.0 + 0.025\ V\sqrt{d}.$$

They use the following equation to determine superelevation on curves:

$$S = \frac{V^2W}{gR}$$

where the additional term:

W = channel width at elevation of centerline water surface.

AMAFCA goes on to say that superelevation plus other disturbances can be ignored if less than 0.5. Calculating the required superelevation for a 100 ft. wide channel, 10' depth, 12 fps velocity and 2000' radius resulted in superelevation of 0.14' for SCS and 0.31' for AMAFCA, both below 0.5 foot.

5.3 Transitions

Transitions in the channel section for box culverts, control structures, stilling basins, etc. are generally a problem for construction due to their varying side slopes and channel widths. Unfortunately, they are hydraulically a necessity since they can control the capacity of the whole system. The hydraulic design of transitions must be dealt with on a site specific basis and are not considered in this study.

Transitions from one width channel to another or from one side slope to another are generally handled with straight line widening and slight warping of the slope paving. These are generally not a major construction problem especially when side slopes are not steeper than 1 1/2:1.

5.3.1 Arizona Department of Transportation

Some of the methods of shaping and constructing transitions have been reviewed and are discussed below. Most of the transitions within the reviewed ADOT system were constructed by warping the side slope from normal to vertical over a distance while widening the invert. Some have thickened the slab plus added reinforcing while others have only added reinforcing to account for the lateral earth pressure as the paving gets steeper. In the case of the Outer Loop, the transitions were designed as retaining walls with varying thickness and varying reinforcing to resist the lateral pressure. The intersection of base and slope wall was, however, a straight line on all transitions.

5.3.2 U.S. Soil Conservation District

The SCS manual[11] shows transitions with not only warped side slopes but also with circular transitions of the invert/side slope intersection and top of wall. These would no doubt function better hydraulically but would be far more difficult to construct. They also indicate the use of straight transitions similar to ADOT's for "Less Important Transitions".

5.3.3 U.S. Army Corps of Engineers

On the ACDC channel, the Corps has used a different approach to transitions which is no doubt simpler to build and accounts for any lateral earth pressure. It, however, may not have hydraulic properties as good as the warped method currently being used by ADOT. They have used a variable thickness vertical retaining wall that follows a straight line (Figure 1) as it widens to meet the width of the culvert, stilling basin, etc. The top of the vertical wall slopes up from zero at the start of the transition to the top of the slope paving where it meets the structure. The slope paving stays at its normal slope and ties into the top of the wall as it gets higher thus eliminating warping.

5.4 Vertical Slope Breaks

One other consideration relating to the hydraulic design of water conveying channels was noted in the Corp's manual "Hydraulic Design of Spillways" (EM 1110-2-1603)[12] and relates to vertical curves. It states, "A break in grade of the profile from a light slope to a steeper slope will cause a negative pressure on the slab unless the vertical curve is properly designed. The design is similar to that for a parabolic drop from a tunnel exit portal to a stilling basin floor. The equation for the trajectory of a jet is given in EM 1110-2-1602, subparagraph 25c. The velocity used for computing the curve should be 25 percent greater than the estimated mean velocity of design flow." For the low velocities that will occur in most linings in the Phoenix area this should not be a major problem. Thickening paving or use of vertical curves to handle this potential negative pressure will require design on a project site specific basis.

6. SUMMARY AND CONCLUSIONS

6.1 Lining Thickness

Review and analysis of the collected material on channel lining criteria as previously discussed indicates the current general practice for establishing base slab thickness versus water velocity follows closely the SCS nomograph except for the Corps, Albuquerque and the Los Angeles area. As previously discussed, the Corps criteria was developed for the L.A. area where high corrasion problems exist. After discussing the design of the Corps' ACDC with Mr. Ford, it was determined that, with the low corrasion potential in Phoenix area, thicknesses of the magnitude recommended by SCS are compatible with the Corps' thinking. It is our conclusion that, subject to other aspects discussed in the reinforcement summary, the SCS nomograph provides a reasonable approach to establishing the bottom slab thickness. Bottom slab thicknesses would therefore be:

VELOCITY (fps)	THICKNESS (inches)
less than 10	4
10 to 15	6
15 to 20	7
more than 20	8

A minimum 6-inch bottom slab, as previously discussed, would be required in any channel wide enough to accommodate maintenance vehicles.

The general practice for side slopes, with the exception of SCS, has been to use a thickness approximately 80 percent of the base slab thickness. To maintain reasonable reinforcement clearance and allow some construction tolerance for fine grading excavation, a minimum thickness of 4 inches is considered prudent. Except as discussed further in the reinforcement summary, following would be the slope lining thickness:

VELOCITY (fps)	THICKNESS (inches)
less than 10	4
10 to 15	5
15 to 20	5 1/2
more than 20	6

6.2 Reinforcement, Joints and Maintenance

Reinforcement criteria is highly dependent on joint spacing and to some degree on maintenance of the channel. The maintenance aspect consists mainly of the problems experienced by most agencies, ADOT included, of maintaining expansion joints. Maintenance of expansion joints was the major complaint heard during our investigation. A second problem relating to joints, as discussed in the Geotechnical Report, is the potential of open joints being filled with incompressible material after shrinkage and during periods of contraction. Expansion during periods of higher temperature will increase the compressive stress and, as discussed previously on SCS's Power Line Floodway, cause local buckling of the lining. Another problem with open expansion joints or widely open contraction joints is infiltration of water into the moisture sensitive soils that are found in a minor part of the Phoenix area. The final problem with joints was observed in the Phoenix area and consists of large weeds growing from the sediment collected in the joints. The standards or criteria for the design of reinforcement falls into two basic categories, excluding the USBR unreinforced canal linings, and is dependent largely on joint spacing.

6.2.1 Continuously Reinforced Channel Lining

The Corps criteria eliminates all joints, both expansion and tooled or weaken plane joints, and uses continuous longitudinal reinforcement of 0.30 to 0.40 percent depending on climatic conditions. Transverse reinforcement is recommended at 0.20 to 0.27 percent. Percentages for the Phoenix area would be 0.30 and 0.20 for longitudinal and transverse respectively. The Corps' experience with the continuous reinforced channel linings has been excellent. With good concrete control the Corps has experienced only hairline cracking at about 12 to 13 foot centers. This approach to design would eliminate the maintenance problems with the joints, reduce the potential problems with moisture-sensitive soil and limit filling of joints with incompressible materials.

The use of 0.30 and 0.20 percent reinforcement does, however, place some limitation on minimum lining thickness. To maintain proper or reasonable reinforcement clearance and allow realistic construction tolerances with two layers of #3 bars or the size of mesh required, a minimum lining thickness of 5 inches will be required. A 5-inch lining would, however, require a deviation from the minimum ACI requirement of 3-inches clearance for reinforcement against grade. Based on the problems experienced with the bulging of the concrete over the reinforcing on the ACDC channel, this is considered a minimum thickness. Since #3 reinforcing has a premium cost for both material and placement, costs were developed for a 5-inch lining with continuous #3 reinforcing (Table 2) to allow comparison with the costs of the 6-inch linings reinforced with #4 rebar. Although the $14.73 per-square-yard cost of the 5-inch slab is less than the other options discussed below, it is our conclusion that based on discussions in the June 16, 1987 report review meeting relative to corrasion problems with less than 3 inch cover and the desire for a 50-year life, the 5-inch thickness will not be adequate for use with tied reinforcement. A minimum 6-inch thickness with #4 reinforcing and 3-inch clearance to grade should be maintained. Should flat mesh reinforcing become competitive, 6 inch paging with W5 wire and 3-inch clearance would be satisfactory. Maintaining a 3-inch clearance in a 4-inch slab with 0.3 percent reinforcement even with mesh is not practical, therefore, 4-inch paving is not recommended.

6.2.2 Discontinuously Reinforced Concrete Lining with Expansion Joints

The second approach to design is the "Subgrade Drag" method which establishes the reinforcement based on expansion joint spacing. The criteria of a number of agencies, as previously discussed, basically follow the "Subgrade Drag" approach when joint spacing and reinforcement is analyzed. Only the California Division of Highways' publication "Bank and Shore Protection in California Highway Practice"(8) actually recommends the use of the formula. Use of the "Subgrade Drag" approach, of course, reintroduces the maintenance problem of expansion joints that have been experienced by most agencies including ADOT.

6.2.3 Cost Comparison

To better compare continuous vs. discontinuous reinforcement, costs were developed for a 6-inch lining by both criteria. Since the minimum reinforcing by pre-1980 SCS subgrade drag criteria was 0.10 percent, this was used to establish the required minimum steel and expansion joints spacing. Based on SCS's shrinkage and temperature reinforcing steel graph, (Fig. A-2, Appendix B) the maximum joint spacing would be 40 feet with #3 @ 18" each way. The resulting average cost including concrete, reinforcing (with premium for #3 rebar) and expansion joint is $14.54 per square yard (Table 2). For a continuously reinforced 6-inch lining, reinforcement would be #4 @ 11" longitudinal and #4 @ 16" transverse. The average cost for the continuously reinforced slab is $15.28 per square yard.

To provide a full cost comparison of design approaches, costs were also developed for a 6-inch lining by the current SCS 0.3 percent and 0.5 percent reinforcing requirements. The minimum 0.3 percent requirement results in #4 @ 11" longitudinal with expansion joints at 30-foot centers. The average cost with joints at 30-foot centers is $16.89 per square yard (Table 2). For joint spacing greater than 30 feet, the 0.5 percent longitudinal reinforcement (#5 @ 10") would permit transverse joints at 180-foot centers per the subgrade drag formula. The cost for a 6-inch lining with maximum joint spacing and 0.5 percent reinforcement would be $18.25 per square yard (Table 2).

None of the cost include excavation or fine grading which would be the same. The cost for a 6-inch lining with discontinuous reinforcement and expansion joints, therefore, ranges from $14.54 to $18.25 per square yard.

6.2.4 Conclusions

Although there would be a cost savings of $0.74 per square yard by using expansion joints at 40-foot centers, (fully sealed joints and not tooled or weaken plane joints) it is our opinion that the savings would be spent in maintenance over a few years. Increasing the joint spacing and reinforcement, as seen, very quickly increases the construction costs above that of the continuously reinforced lining to a point where the cost of the SCS lining with 0.5 percent reinforcement exceeds the continuous lining cost by $2.97 per square yard. Based on this and previous discussions on maintenance and joint considerations, it is our conclusion that the continuously reinforced lining will provide a more cost-effective and serviceable channel.

6.3 Arizona Department Channel Lining Reinforcement

During our review of accumulated materials and discussions with various agencies, we have encountered a number of channel lining standards that use relatively light reinforcing with only contraction (weakened plane) joints at 15' to 20' spacing. Expansion joints, if used at all, are widely spaced. The Superstition channel falls into this category and to a degree so does the Ehrenberg channel since no expansion joints are used. The reinforcing, however, in the Ehrenberg Lining is fairly heavy but tooled joints are at 20-foot centers. Since our inspection of the Phoenix channels did not reveal any major cracking away from the joints; we questioned how they were performing.

6.3.1 Superstition Freeway Channel

We therefore reviewed the performance of the Superstition channel based on the subgrade drag formula allowing the coefficient of subgrade drag to vary from the general ranges of 1.5 to 2 given in Design of Slabs on Grade.[5] Analysis of the Superstition channel indicated the 6x6-10/10 mesh used in the 4" lining was barely adequate at normal stresses to resist cracking between the 20' joints. Further analysis indicated that since no expansion joints were used, the mesh could reach yield strength at 38 to 50-foot centers or about every second to third joint if no cracking occurred at intervening joints. This yielding would allow joint opening as shrinkage occurred thereby creating an expansion joint. Since some cracking will occur at every joint, yield more realisticly will occur every 4th to 6th joint. Width might be of the order of 1/8" to 3/16".

6.3.2 Ehrenberg-Phoenix Highway Channel

A similar analysis of the Ehrenberg lining with the W10xW10 mesh, of course, indicated no problems with cracking between tooled joints which is agreeable with our observations. It was determined that yield could occur at 130 to 170 feet or every 6 to 9 joints if no cracking occurred at intervening joints. Again, this yielding during shrinkage would allow the slab to create its own expansion joint. Again, since some cracking will occur at every joint, yield more realisticly will occur in the range of every 8th to 12th joint. Joint width where yielding occurs might be on the order of 1/4 to 3/8 inch. This agrees with Dan Lance's reported opening of occasional cracks in the order of 3/8 inch. It would also be in keeping with our observance of large weeds growing in the sediment at the joints.

This led us to the conclusion that, when the linings are constructed with either the continuous light or heavy mesh, without expansion joints but with tooled (weakened plane) joints, the linings create expansion joints at various intervals at the weakened planes during curing and shrinkage. This was discussed with Mr. Ken Hanson, Denver PCA representative, and Mr. Paul Muller, former Phoenix PCA representative and currently at Arizona State University. They both concurred with this thinking.

6.3.4 Power Line Floodway

Quite late in our program, we were able to contact Mr. Ralph Arrington, Arizona State Engineer, SCS and obtain the information already discussed on the SCS Power Line Floodway near Phoenix. Although expansion joints were used in its construction, they were widely spaced (325') and tooled joints included at 50-foot intervals between expansion joints. Since only 0.10 percent steel was used, the design is similar to Superstition and was not designed using SCS's current nor the subgrade drag criteria.

Analyzing the channel by the subgrade drag formula indicates that yield in the 6x6-6/6 mesh could occur at 60 to 80-foot centers or almost every tooled joint. Although crack widths were not discussed with Mr. Arrington, he indicated that joints had cracked, opened and filled with dust, thereby causing the spalling and buckling due to the temperature expansion. This lends further credence to our conclusion of how the channel linings function when constructed with weakened plane joints.

It also points out that a potential buckling problem exists on the Phoenix channels where joints have filled with dust or sediment.

6.4 Alternate Design Criteria

Based on the review and analysis of the Superstition, Ehrenberg and Power Line channels, we have evolved an alternate approach to reinforcement design that deserves some consideration. The approach would involve a certain degree of risk and increase the maintenance cost over that of a continuously reinforced lining. It would result in a moderate savings in original construction cost.

The approach would use 1-1/2" tooled joints at 30-foot centers with the reinforcing designed by the subgrade drag formula to control cracking between joints. Reinforcing would be made continuous and the lining allowed to generate its own expansion joints where yield occurs in the steel. After shrinkage and cracking occurs, all major cracks would be filled with sealant to prevent leakage an infiltration of dust or sediment. This would require a program of regular inspection to find any new cracks and insure that sealant has not been damaged or dislodged.

6.4.1 Cost Comparison

For a 6-inch lining thickness and 30-foot joint spacing each way, the reinforcement requirement would be 6x6-W6xW6 mesh or equivalent rebar.

Yield would theoretically occur at 80 to 105 feet. Allowing for cracking in intervening joints, yielding would probably be every 5th to 7th joint. Cost for the alternate approach, including sealing every sixth joint and tooled joints at 30-foot centers each way, would be $14.23 per square yard (Table 2). This represents a reduction of $1.05 per square yard from the cost of a continuously reinforced 6-inch lining.

Although this is slightly better than having true expansion joints at 40-foot centers, we again feel the savings will be used for maintenance in a few years. It also has a certain degree of risk if maintenance does not occur.

6.4.2 Conclusion

We offer the approach for consideration but remain with our original conclusion that the continuously reinforced linings without expansion or tooled joints will provide the more serviceable channel. Expansion joints would be used only at the vertical face of bridge piers and abutments. No expansion joints would be installed at culverts, channel control structures, stilling basins, etc.

6.5 Hydraulics

Hydraulics criteria is not part of this study except as it might affect the design and details of the channel lining. General review of ADOT's hydraulic policy indicates it is compatible with current design standards with the possible exception of use of the 50 instead of the 100-year event. We fully concur with this design approach except that in cases where overtopping could cause major loss of life or extensive property damage in heavily populated or developed areas, the 100-year rainfall event with freeboard should be used. A set of Hydraulic Design Guidelines for Drainage Channels would be a great help to design consultants.

Review of current practices for design of channel transitions indicates the warped transitions are not the easiest to construct and may not perform the best with the continuously reinforced linings. It is our conclusion that the vertical wall type transition being used on the Corps' ACDC channel (Figure 1) will provide the best structure. It simplifies construction, resists earth pressure and eliminates potential compressive problems in a warped slab.

Access ramps that slope in an upstream direction cause hydraulic disturbances and should, if possible, be changed to slope downstream. The low flow side of the channel invert should be located on the side away from the access ramp to reduce silting problems. O-Gee control sections in the channels could be constructed with approximately 30 to 50 percent opening along the channel floor to flush sediments through during low flows with out a major disturbance in the O-Gee characteristics.

6.6 Concrete

Concrete strengths in the reviewed standards vary from 3000 psi to 5000 psi in the Los Angeles area where high corrasion potential exists. The major concern for continuously reinforced lining is shrinkage and crack control. Channels with maintenance traffic will require a concrete that has a Modulus of Rupture (flexural strength) of 475 psi. The interrelated effects of temperature, water and cement content, aggregate, and construction practices on drying shrinkage, warping and curling are discussed in detail in the Geotechnical and Engineering Materials Report by Sergent, Hauskins & Beckwith, Appendix A, along with their conclusions and recommendations.

6.7 Cutoff Walls

Cutoff walls have generally been used in channel linings to eliminate progressive failure. Other than SCS, this practice has been discontinued except at the beginning and end of channels. The SCS, however, is not using cutoff walls on the RWCD channel. It is our conclusion that cutoff or stabilizing walls are not required to prevent progressive failure in continuously reinforced channels, however, there is some concern for the stability of the lining slope walls at transitions where the channel starts to widen and for the channel invert at breaks in slope. The change in direction will result in an unbalanced force acting away from the supporting earth. To prevent local buckling at these locations, it is our conclusion that cutoff or stabilizing walls, rigidly attached to the paving, should be installed to stiffen the linings. Stabilizing walls will also be required at the start and end of channel where they change to riprapped or other types of lining and at existing structures where the new lining can not realisticly be made continuous with the existing lining. In addition, stabilizing walls are recommended immediately upstream and downstream of movement sensitive structures such as intersecting drainage channels.

6.8 Subgrade Treatment, Drainage, Pressure Relief and Angle of Side Slopes

The angle of repose of the soil, required subgrade treatment, drainage, and pressure relief requirements are discussed in the Geotechnical Report, Appendix A. Although some of the soils in the Phoenix area can stand at 1:1 slopes, it is our opinion that channel linings should be constructed with maximum slopes of 1.5:1.

6.9 Details

Most of the construction details used on the ADOT channels are quite satisfactory. Of course all joints, except construction joints with continuous reinforcement, will be eliminated. The width of the paving at the top of the slope could be narrowed but needs to remain wide enough to allow machine trenching of the top cutoff wall. If heavy gage sheet mesh is offered as an alternate to grade 60 reinforcing steel, the length of the side slopes and profile of the top edge must remain constant. Currently there is little potential that flat sheets will be competitive with standard rebar.

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TABLE 1

AGENCY	VELOCITY (f/s) OR FLOW (cfs)	LINING THICKNESS (Inches) SLOPES	LINING THICKNESS (Inches) INVERT	REINFORCEMENT SIZE & SPC.	REINFORCEMENT % Ac	SIDE SLOPES
U.S. Bureau of Reclamation	<500 cfs 500 to 2000 cfs 2000 to 5000 cfs Velocity > 8 f/s not recommended	3 1/2" 4" 4"	3 1/2" 4" 4"		0.10 0.30 0.40	1 1/2:1
U.S. Army Corps of Engineers (ETL 1110 - 2-236)	Small side Channels Main Channel Paving	not < 4" 8", not <6"	not < 6" 10", not <8"	Trans. Long.	0.20 to 0.27 0.30 to 0.40	--
ACDC	23,000 cfs 4 to 15 f/s	6"	8"	#5 @ 18" #5 @ 15"	0.30	2:1
Soil Conservation Service	10 fps 15 fps 20 fps (Determined by Nomograph)	4" approx. 6" approx. 8" approx.	4" approx. 6" approx. 8" approx.		0.30 0.50	2:1
RWCD Channel	5,600 cfs	6"	6"	#5@10"	0.50	2:1
Caltrans (1987)	<10 fps 10 to 15 fps >15 fps	3" - 3 1/2" 4" - 5" 6"+	3 1/2" - 4" 5" - 6" 7"+	#3 @ 18" #3 @ 15" #3 @ 12"	0.146 0.125 0.131	Angle of repose of soil or flatter
California Division of Highways (Bank & Shore Protection in California Highway Practice, 1970)	Light Duty Slope Paving Heavy Duty Slope Paving	3" or 4" 4" or 6"	--	Reinforcement designed by subgrade drag formula		1 1/2:1 or flatter
Los Angeles Flood Control District	--	6" min.	8" min	--	0.18	1 1/2:1
San Diego (1983)	Minor Drainage Channel 8' max. bottom width Major Drainage Channel <8' bottom width	4" 4"	4" 6"	6x6xW1.4xW1.4 minimum 6x6xW1.4xW1.4 minimum		1 1/2:1
Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA, 1981)	<25 fps >25 fps	6" 6"	7" 8"	Long. Trans.	0.50 0.25	--

TABLE 1 (CONTINUED)

AGENCY	CONCRETE PROPERTIES	JOINT SPACING		CUT-OFF WALL SPACING	WEEP HOLES
		CONTROL	EXPANSION		
U.S. Bureau of Reclamation	f'c = 3000 psi to 3750 psi	12 to 15' 30' 30'	None	At structures and at ends of transitions for structures	None
U.S. Army Corps of Engineers (ETL 1110 - 2-236)	f'c = 3000 psi	None	1/2" at structures with waterstops	None	Closed drainage system when invert is below water table.
ACDC	f'c = 3000 psi 0.52 w/c ratio, 1 to 3" slump, 6% air	None	At existing structures with sealant	None, except at start and end of lining	None
Soil Conservation Service	--	None	≤30 feet >30 feet	At structures and not to exceed 1000'	
RWCD Channel	--	None	100 feet	None	--
Caltrans (1987)	--	None	20' spacing indicated for slope paving and in specifications for lining	20' spacing indicated for slope paving	Subsurface drains and weep holes when hydrostatic pressure is anticipated
California Division of Highways (Bank & Shore Protection in California Highway Practice, 1970)	6 sacks/ c.y.	--	1/4" joints at 60'± for light and at 20 to 30' for heavy duty	20 to 30 feet	6' horizontal and 10' vertical spacing
Los Angeles Flood Control District	f'c = 5000 psi invert for heavy bed load, otherwise 4000. 4000 or 3250 psi side slopes.	No transverse or longitudinal control joints	50' centers	None	Using backflow values instead of weep holes
San Diego (1983)	f'c = 3000 psi	Weaken plane joints @ 12' to 15' with joint filler	Expansion joints @ change in section and ends of curves	At each end of channel	4" dia. @ 10'
Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA, 1981)	5.5 sacks/c.y., 0.53 w/c ratio, 3" max. slump, f'c = 3000 psi, 4 to 7% air	None	Expansion joints at rigid structures or to existing lining that is not cont. reinforcing.	Cut-off walls at start and end of lining	--

TABLE 2

CHANNEL LINING COST COMPARISON

6" Lining with Minimum Discontinuous Reinforcement:

As = 0.001x6x12 = 0.07 sq. in.
Use #3 @ 18" e.w.
St. = 0.376x12 = 4.51 lb./s.y.
Expansion Joint Spacing per Fig. A-2 = 40'

Average Cost

Expansion Joint = $5.38/ft.x3	= $	16.14
Reinforcing = 4.51x13.33x$0.55*	= $	33.07
*Premium cost for #3		
Concrete = $10.85 x 13.33	= $	144.67
Total (40' Length)	$	193.88
Average = 193.88/13.33	$	14.54 per s.y.

6" Lining with Continuous Reinforcement

Long. As = 0.003x72 = 0.22 sq. in.
Use #4 @ 11"
Trans. As = 0.002x72 = 0.14 sq. in.
Use #4 @ 16"
Wt. = 0.668x3(3.27+2.25) - 11.07 lb./s.y.

Average Cost

Reinforcing Steel = 11.07 x $0.40	= $	4.45
Concrete	= $	10.85
Total	= $	15.28 per s.y.

6" Lining with Expansion Joints at 30 feet & Discontinuous Reinforcing Equal to 0.3 Percent:

Reinforcement Same as Continuous Lining.

Average Cost

Expansion Joint = $5.38/ft.x3	= $	16.14
Reinforcing = 11.07x10.0x$0.40	= $	44.28
Concrete = $10.85x10.0	= $	108.50
Total (30' Length)	= $	168.92
Average = 168.92/10	$	16.89 per s.y.

TABLE 2 (Continued)

CHANNEL LINING COST COMPARISON

6" Lining with 0.5 Percent Discontinuous Reinforcing and Maximum Expansion Joint Spacing:

Long. As = 0.005x72 = 0.36 sq. in.
Use #5 @ 10"
Trans. As = 0.003x72 = 0.22 sq. in.
Use #4 @ 11"
Wt. = 0.668x3x3.27+1.043x3x3.6 = 17.82 lb./s.y.

Expansion Joint Spacing per Fig. A-2 = 180'

Average Cost

Expansion Joint = $5.38/ft.x3	= $ 16.14
Reinforcing = 17.82 x 60x$0.40	= $ 427.68
Concrete = $10.85x60	= $ 651.00
Total (180' Length)	= $1,094.82
Average = $1,094.82/60	= $ 18.25 per s.y.

6" Lining with Tooled Joints at 30' Centers:

Required As = 0.12 sq. in.
Use 6x6-W6xW6 Mesh or #4 @ 18"
Joint Filler at 180'± centers

Average Cost

Concrete	= $ 10.85
Reinforcing = 0.668x12x$0.40	= $ 3.21
Tooled Joint = 2x3x$0.20/60	= $ .02
Joint Filler = $2.33x3/60	= $ .12
Backer Rod = $0.53x3/60	= $ .03
Total	= $ 14.23 per s.y.

TABLE 2 (Continued)

CHANNEL LINING COST COMPARISON

5" Lining with Continuous Reinforcement:

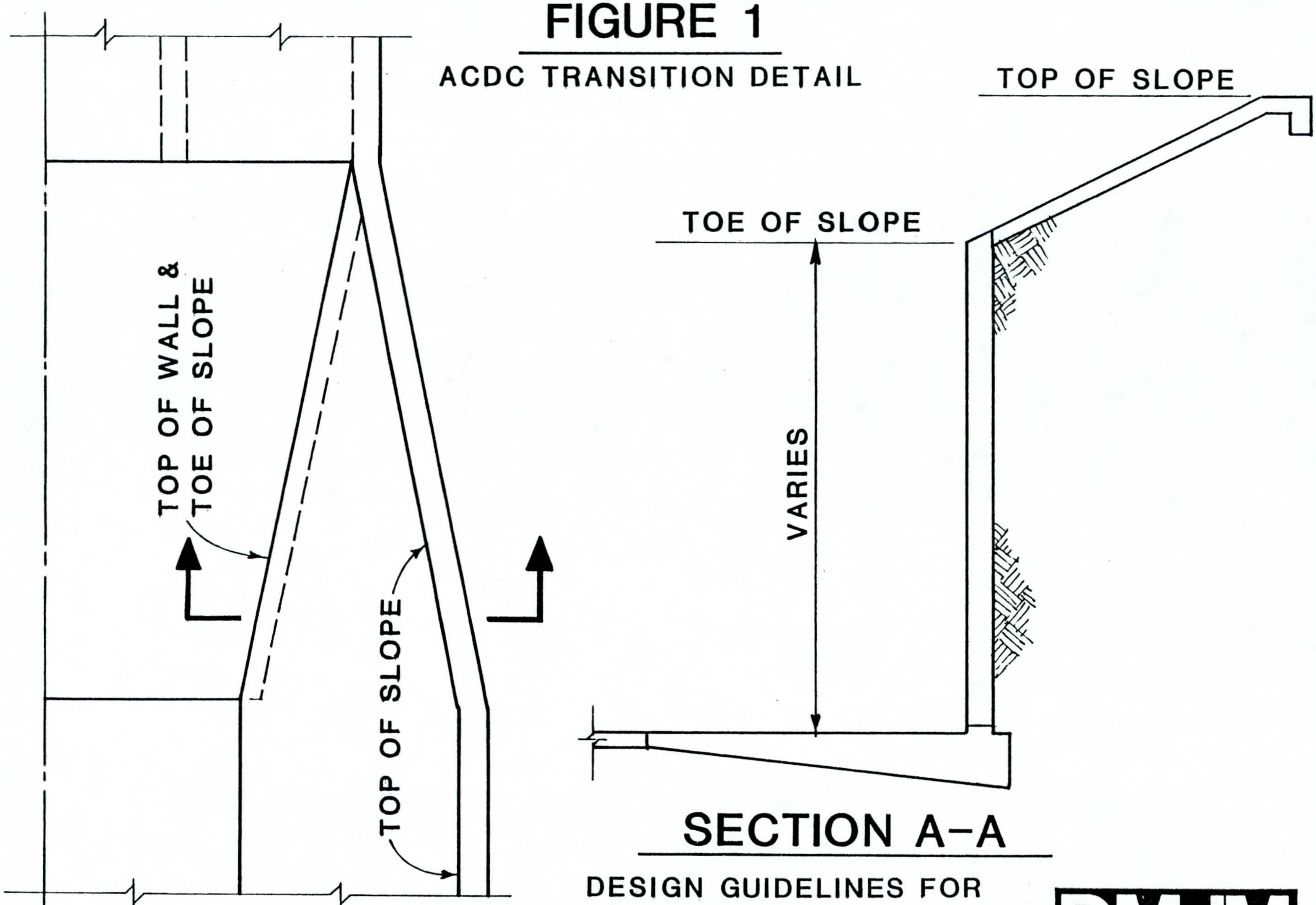
Long. As = 0.003x60 = 0.18 sq. in.
Use #3 @ 7-1/2"
Trans. As = 0.002x60 = 0.12 sq. in.
Use #3 @ 11"
Wt. = 0.376x3(4.80+3.27) = 9.11 lb./s.y.

Average Cost

Reinforcing Steel = 9.11x$0.55*	= $	5.01
*Premium cost for #3		
Concrete = $10.85 - $1.13**	= $	9.72
**Materials cost for one inch of concrete		
Total	= $	14.73 per s.y.

FIGURE 1

ACDC TRANSITION DETAIL



SECTION A-A

DESIGN GUIDELINES FOR CONCRETE LINED DRAINAGE CHANNELS

DMJM

APPENDIX A

SERGENT, HAUSKINS & BECKWITH

GEOTECHNICAL INVESTIGATION REPORT
Design Guidelines for
Concrete Lined Drainage Channels
Arizona Department of Transportation
Phoenix, Arizona

SHB Job No. E87-112

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JAMES R. FAHY, P.E.
MICHAEL HULPKE, P.G.
DAVID E. PETERSON, P.G.
ALBERT C. RUCKMAN, P.E.
PAUL KAPLAN, P.E.

October 23, 1987

DMJM Phillips-Reister-Haley, Inc.
910 Fifteenth Street
Suite 700
Denver, Colorado 80202

SHB Job No. E87-112

Attention: James M. Tolle, P.E.
Associate Vice President

Re: Design Guidelines for
Concrete Lined Drainage Channels
Arizona Department of Transportation
Phoenix, Arizona

Gentlemen:

Our Geotechnical and Materials Engineering Report presenting guidelines for the design of concrete lined drainage channels is herewith submitted. The report includes a discussion of environmental effects on soil-slab interaction, recommendations for the identification and analysis of collapsing and expansive soils, treatment of moisture sensitive soils and other geotechnical criteria, and recommendations for concrete mix designs.

Should any questions arise concerning this report, we would be pleased to discuss them with you.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

Registered Professional Engineer (Civil)
CERTIFICATE NO. 5633
GEORGE HENRY BECKWITH
Date Signed
ARIZONA, U.S.A.

By George H. Beckwith

George H. Beckwith, P.E.

Copies: Addressee (1)
DMJM (5)

REPLY TO: 3232 W. VIRGINIA, PHOENIX, ARIZONA 85009

PHOENIX (602) 272-6848
TUCSON (602) 792-2779
ALBUQUERQUE (505) 884-0950
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SHB Job No. E87-112

Registered Professional Engineer (Civil)
CERTIFICATE NO.
5533
GEORGE HENRY BECKWITH
Date Signed 10/23/87
ARIZONA, U.S.A.

1. INTRODUCTION

This report presents an evaluation of geotechnical and materials engineering considerations for the design of concrete lined drainage channels in the greater Phoenix area. A description of typical soil conditions and methods for evaluating potential ground movements are presented along with criteria for soil-slab interaction analysis, drainage, pressure relief, slope angles, earth pressures against retaining walls and structural design of slabs subject to wheel loads. Environmental effects on slab performance are reviewed and recommendations for concrete mixes are presented.

Both continuously reinforced slabs and slabs with joints with various degrees of reinforcement and spacing of joints are considered.

Slab design is a soil-structure interaction problem wherein the potential for ground movements of moisture sensitive soils and the buildup of hydrostatic pressures below slabs during rapid drawdown must sometimes be considered. The interrelated effects of temperature, drying shrinkage, warping or curling, and their influence on soil-structure interaction and slab design are discussed in Section 3.

As discussed in Section 2, most soils in the Phoenix area are not severely moisture sensitive and provide relatively stiff subgrade support. Thus, the majority of sites require minimal subgrade treatment. A small minority of sites are underlain by significant thicknesses of expansive or collapsing soils which can potentially cause relatively large movements. For these sites, the calculated risk of the soils becoming wet should be considered in determining what subgrade measures and/or structural treatment is appropriate. A similar calculated risk approach is applicable to evaluating the need for pressure relief.

This report applies to the greater Phoenix area but could be expanded to cover the entire state with minor additions. These would include consideration of frost action and different environmental effects at higher elevations, and special measures for sites involving large thicknesses of very highly expansive and collapsing deposits.

2. GEOTECHNICAL CONDITIONS

With the exception of thin zones at the surface which can be treated with ordinary surface compaction, most soils in the study area are relatively firm or dense calcareous alluvial deposits (Beckwith and Hansen, 1982; Crossley and Beckwith, 1978)* or sands and gravels. These predominant soil types are generally not susceptible to appreciable volume changes upon wetting and

*References are listed at end of report.

provide good foundations for concrete slabs-on-grade. Structural fills constructed from these soils will also provide good support for concrete drainage channel linings.

2.1 Expansive & Collapsing Soils

Expansive and collapsing soils which sometimes produce several inches of ground movements are of most concern in the design of lined channels. For example Holtz (1959) gives several examples of severe damage to concrete canal linings including the Welton-Mohawk Canal in Arizona. These types of soils only make up a small minority of the land surface in the study area. However, expansive and collapsing soils of large enough thickness to create serious problems with most slabs are erratically distributed over the greater Phoenix area. Collapsing soils are the most widely distributed and, when subjected to wetting, sometimes have the potential for creating large ground movements under their own weight or low superimposed stresses such as those that might be involved with small drainage structures.

The collapsing soils of the Southwest are predominantly low gradient alluvial fan mudflow-debris flow deposits and alluvial deposits in the floodplains of intermittent streams (Beckwith, 1979). The sedimentation process for alluvial fans involves alluvium deposited in the channels and overbank areas of the small drainage ways on the fan surface and occasional thick mudflows or debris flows which cover the entire surface. In this depositional sequence, each layer is laid down by a flash flood or sheet flow. In the dry, hot climate for which annual evapotranspiration greatly exceeds rainfall, each layer dries out before the overlying layer is deposited. Sedimentation of the next layer wets only the surface of the underlying material. In this way, the deposit, as a whole, is built up without being consolidated under its own weight at high moisture contents.

These soils are geologically young, having been laid down in the last 10,000 to 12,000 years since the last episode of continental glaciation during which the present type of semiarid environment has prevailed. In the environment of the Phoenix area, with little vegetative cover, collapsing soil deposits can usually be identified by analysis of geomorphology by photogeologic techniques.

An example of serious settlements and cracking of an embankment due to wetting of collapsing foundation soils was McMicken Dam (Hansen and others, 1983; Deatherage and others, 1986) north of Luke Air Force Base and west of U.S. 60 (Grand Avenue Expressway). This 9-mile long flood control structure is about 28 feet high and is underlain by up to 15 feet of collapsing soils. It settled several inches in many areas and developed hundreds of transverse cracks up to about 3/4 inch wide. These ground movements were sufficient to damage most types of concrete linings.

2.2 Shrinkage Cracking of Embankments

The U.S. Soil Conservation Service (Sterns and others, 1978) reports shrinkage cracks which developed in embankments with only a low to moderate shrinkage potential. These dams rested on firm foundations which were not moisture sensitive. These embankments which are composed primarily of silty and clayey sands and gravels developed cracks 1/2 to 3/4 inch wide at the surface extending to about 10 feet. This phenomenon has been identified in other embankments in the Phoenix area by this firm. The U.S. Soil Conservation Service structures involved included the White Tanks No. 3 Dam along I-10 west of Phoenix and the Rittenhouse, Vineyard Road and Magma Dams south of the Superstition Freeway (SR 360) in the Apache Junction area. These shrinkage movements of the soil are sufficient to impose large stresses on concrete linings on the face of embankments.

2.3 Types of Geotechnical Profile

Geotechnical conditions can be adequately defined for purposes of design in terms of the five cases shown in Table 1. A discussion of each subgrade condition is provided in the following sections. This categorization is based on experience gained by this firm in the performance of about 4,000 geotechnical investigations in the Phoenix metropolitan area over the last 28 years. It is anticipated that in many cases, channels will be constructed by cutting and building dikes above grade so that the concrete linings will be partly on cut surfaces and partly on fill. The following discussion applies both to the native soil subgrades and fills constructed from borrowing the native soils.

2.3.1 <u>Case 1: Clean Sands or Sands & Gravels</u>

Subgrade conditions consisting entirely of relatively dense clean sands and gravel mixtures occur within the floodplains of the Salt, Gila and Agua Fria Rivers and some other drainages. These soils are generally poorly graded, angular to subrounded, nonplastic and dense to very dense, and sometimes contain large amounts of cobbles and boulders. They provide excellent foundation support and require only minimal subgrade preparation prior to channel construction. They are relatively free draining.

2.3.2 <u>Case 2: Cemented Desert Alluvium</u>

Moderately to strongly cemented alluvial soils which are not significantly moisture sensitive are wide-spread across the greater Phoenix area and often are present over the full depth of the profile. Often cemented alluvium consists of predominantly finer grained soils composed of varying clay, sand and silt mixtures with occasional gravels. In other instances, the cemented alluvium is coarser granular soils with considerable silt and clay. These deposits are moderately to highly stratified, low to medium in plasticity and firm to hard at low moisture contents. Beckwith and Hansen (1982) describe these deposits as Class 2 and 3 calcareous soils. Calcium carbonate is the predominant cementing agent with occasional coformation of silica, magnesium or calcium sulfate. These materials generally provide good foundation support for channel construction. Foundation treatment for embankments can be accomplished with ordinary surface compaction. The soils are not free draining.

An example of these deposits is found along most of the Superstition Freeway alignment through the Tempe and Mesa areas.

2.3.3 Case 3: Moisture Sensitive Collapsing or Expansive Soils Over Poorly Drained, Cemented Desert Alluvium

Case 3 involves a layer of moisture sensitive surface soils overlying the types of poor draining, cemented soils described for Case 2. Foundation treatment alternatives and associated calculated risks in employing each alternative depend on the thickness of the moisture sensitive soils.

For collapsing soils, the upper moisture sensitive soils are primarily silty sands, sandy silts and clayey sands of low plasticity. Lesser stratifications of clean sands and lean clays are often present. The soils are predominantly low-gradient alluvial fan deposits and alluvial deposits in the floodplains of intermittent streams formed during the Holocene Epoch (within the past 10,000 to 12,000 years).

Generally, only minor clay development and calcareous cementation are present in these deposits. Identification techniques and foundation treatment alternatives for these soils are discussed in Section 4.

Case 3 conditions with collapsing soils occur along the Superstition Freeway (SR 360) from Ellsworth Road to U.S. 60/89, the Outer Loop (SR 117) in the Scottsdale and Paradise Valley areas and the Grand Avenue Expressway northwest of Sun City. Extensive parts of SR 517 (Cotton Lane and Northwest Loop) are expected to be underlain by Case 3 conditions.

A Case 3 example involving expansive surface soils covers portions of the Outer Loop near the intersection of I-17 where a thin layer of highly expansive soils at the surface is underlain by cemented, clayey, granular soils.

2.3.4 Case 4: Moisture Sensitive Soils Over Free Draining Granular Strata

Case 4 involves a layer of the types of collapsing or expansive soils described for Case 3 overlying free draining, relatively dense granular soils of the type involved in Case 1. Examples are collapsing silt-fine sand deposits in the Salt and Agua Fria River floodplains overlying sand and gravel.

2.3.5 Case 5: Deep Expansive Clays

Case 5 involves medium to highly expansive clays to sufficient depth that it is usually not feasible to remove and replace them with nonexpansive soils. Usually, this case applies to clays 5 to 10 feet in depth or more. Typically, these deposits which consist of sandy to silty clays are weakly to strongly cemented, medium to high in plasticity and firm to hard at low moisture contents. Upon wetting of these soils, significant expansion will occur under low confining stress, such as below the lined channel side slopes and floors.

An example of Case 5 is portions of the Price Expressway (SR 117) where medium to highly expansive clays will form the entire profile in the upper 15 feet.

Identification techniques and foundation treatment alternatives for expansive soils are discussed in Section 4.

2.4 Groundwater Conditions

Over most of the study area, the groundwater table is well below the elevation that could affect lined channels. However, in portions of the Salt River channel downstream of the Tempe Bridge, the water table could potentially rise above the bottom of channel slabs during long-term flows. Pressure relief needs to be considered in design in these cases even if the subgrade is free draining. In a few areas, shallow, perched water tables have developed due to heavy irrigation which could rise after construction. This condition occurs for part of the Southwest Loop south of the Salt River.

3. INTERRELATED EFFECTS OF TEMPERATURE, DRYING SHRINKAGE & CURLING ON SLAB PERFORMANCE

The interaction of slabs and the foundation soils is profoundly influenced by the interrelated effects of temperature, humidity and drying shrinkage which often produce curling and separation of slabs from the subgrade at unrestrained edges and joints (Spears, 1978; Anchor, 1981; Chandler, 1982; Kraai, 1982; Transportation Research Board, 1979, 1982; Ytterberg, 1987a, 1987b, 1987c; Leonards and Harr, 1959; Reddy and others, 1963). Although these factors are important in all environments, they are of particularly great significance in the Phoenix area where wide variations in ambient temperature create relatively large joint movements and slab stresses. These wide temperature variations in combination with low atmospheric humidity combine to produce high thermal and moisture gradients across slabs which create curling or warping. Cracks caused by thermal expansion and contraction, and drying shrinkage can provide pathways for high rates of water infiltration into moisture sensitive soils with consequent subgrade movements. This can lead to progressive damage from thermal stresses, loss of aggregate interlock at joints and cracks with increased curling and progressive ground movements as the opportunity for water to seep into the supporting soils increases.

The effects of temperature and drying shrinkage are influenced by concrete mixes and construction procedures. Their effects can be reduced, but not eliminated, with good mix designs and construction procedures. Effects can be substantially reduced by reinforcement.

3.1 Temperature Variations

The wide temperature variations in the local environment impose severe effects on slabs with open joints or cracks which may be initially caused by soil movements, drying shrinkage or structural failure of curled edges which are separated from the subgrade.

As stated in Synthesis of Highway Practice 56 (Transportation Research Board, 1979):

> "Jointed pavements have an inherent deficiency: Joint spaces are susceptible to infiltration of incompressible material during periods of pavement contraction. Over a period of time, this process may usurp whatever expansion space was built into the pavement initially, with the result that high compressive stresses are produced during periods of greatest pavement expansion, such as when temperature and moisture content are high."

In the Phoenix area environment, large amounts of desert dust (Pewe and others, 1981) and dune sands (Bagnold, 1941) are available which can rapidly fill open joints. The hot, dry climate tends to create relatively fast weathering and deterioration of joint filling materials (Transportation Research Board, 1982).

An example of this phenomenon is described by Morris (1987). Plain concrete pavements for I-10 and I-17 through Phoenix were completed in the early to mid 1960's and had saw cut joints at spacings varying from 13 to 17 feet. Filling of joints resulted in large forces being exerted against abutments of several bridges. Assessment of abutment movements and deflection of bridge girders, and analysis of joint openings upon relief of stresses at the abutments, indicated that concrete compressive stresses of about 1,000 psi had built up. Joint openings occurred for about 900 feet away from the bridges after relief of stresses. Instruments placed in some of these pavements indicated compressive stresses of about 300 psi built in the first year.

3.2 Drying Shrinkage

Ytterberg (1987a) provides a comprehensive review of the factors influencing drying shrinkage. Drying shrinkage is the reduction of concrete volume resulting from the loss of water after hardening, and is caused principally by contraction of the calcium silicate gel as moisture not needed for hydration is lost. It is a major factor in curling as discussed in Section 3.3.

Shrinkage is mostly related to the water demand of the constituent materials and can be reduced by controlling that demand. Selection of coarse aggregate can affect shrinkage almost 100 percent from lowest to highest. The hard, clean durable aggregates from the Salt River and similar sources in the Phoenix area have relatively low shrinkage potential.

Coarser, low alkali Type II cement produces low shrinkage concrete paste as compared to higher early strength Type I and Type III cements. Cement content and, thereby, shrinkage can be reduced by using coarser, well graded aggregates with the maximum size being up to one-third the slab thickness.

A widely held misconception is that shrinkage can be controlled simply by using low slump mixes. Ytterberg (1987a) recommends shrinkage tests of concrete by the method developed by Kraai (1985) to aid in developing mixes with specified maximum acceptable water content, cement content and water-cement ratio. Specified compressive strength should be 3,000 psi and should be required at the latest possible age.

Abrasion resistance of slab surfaces is not closely related to compressive strength, but is a function of water-cement ratio and aggregate quality, which is controlled by the amount and quality of surface finishing. Accordingly, it is not necessary to specify high strength to achieve good resistance to wear.

According to Ytterberg (1987a), the three-day minimum strength requirement in ACI 302 severely limits the use of low shrinkage Type II cement and leads to higher shrinkage concrete. Also, the requirements for 3-inch minimum slump in ACI 302 forces the use of high-range water reducers which may increase rather than decrease shrinkage.

Construction practices such as allowing the concrete to reach high temperatures and long haul times in the transit mixer with excessive revolutions at mixing speed can increase shrinkage.

3.3 Warping & Curling

As discussed by Ytterberg (1987b), warping or curling of concrete slabs on grade is caused by differential moisture and temperature gradients across the slabs, or a combination of both. Edge lift from the subgrade due to curling is common in the Phoenix area and appears to be caused by very high moisture gradients due to high temperatures and low humidity. Nearly impervious subgrades found at many sites increase the moisture gradients and the degree of differential drying shrinkage.

In the Phoenix area environment, curling is probably most extreme during periods of cooling when temperatures are higher at the bottom of the slabs than the top. In some environments, downward curling of slab edges can occur during times that surface temperatures are higher than those at the bottom. In the Phoenix area with very high moisture gradients, curling appears to be confined to edge lift.

Curling may be suppressed during the curing process, but will occur when the surface of the slab is exposed to low humidity. Curling can be minimized with the use of low shrinkage concrete and relatively heavy reinforcement. Extremely heavy reinforcement is needed to eliminate curling (Ytterberg, 1987c).

Wet subgrades will increase moisture gradients and curling. Thus, the subgrade should not be wetted prior to placement of concrete. Permeable subgrades reduce moisture gradients and, thereby, curling to some degree.

The use of permeable granular bases when subgrades are low in permeability will at least slightly reduce curling. However, the benefits of granular base do not appear to be great enough to justify their use for drainage channel slabs in the Phoenix area.

Leonards and Harr (1959), and Reddy and others (1963) present stress and deformation analysis of slabs subject to curling and relate moisture gradients to equivalent thermal gradients. Their analysis shows that there is a critical slab width beyond which the amount of edge curling will not increase. This appears to be in the range of 25 to 30 feet for typical Phoenix area conditions.

During curing of edges, load is transferred to the center of the slab which deflects, or settles into the subgrade. The very stiff subgrades which prevail in the Phoenix area result in less settlement at the center, and a greater width of separation of the edge of slabs from the subgrade. Leonards and Harr (1959) show that as much as 8 feet of the edge the slab can separate from the subgrade for unreinforced slabs with higher shrinkage concrete. The separated parts of slabs are, of course, susceptible to failure under load.

Leonards and Harr (1959), and Reddy and others (1963) also show that stresses created by curling increase with increased modulus of elasticity of concrete. The hard aggregates used in the Phoenix area result in relatively high moduli of elasticity of concretes and, therefore, relatively high bending stresses.

4. CONCLUSIONS & RECOMMENDATIONS

It is concluded that continuously reinforced concrete slabs will provide the best performance and lowest maintenance for the environment of the Phoenix area. It is recommended that special low shrinkage concrete be used for the slabs. Guidelines for low drying shrinkage mixes are given in Section 4.7 and bable 6. A program of trial mixes and field shrinkage tests are recommended to provide reference criteria for development of mixes with the lowest practicable drying shrinkage for each project.

It appears that crack widths for slabs with relatively heavily reinforcement and low shrinkage concrete can be limited to the range of 0.003 to 0.03 inch. These tightly closed cracks will maintain effective aggregate interlock. Not only is the potential for distress of joints and the related high maintenance avoided with the use of continuously reinforced slabs, but the slabs have much lower leakage rates. Thus, with continuous reinforcement, there is less potential for volume changes in moisture sensitive soils and less need to provide pressure relief beneath slabs. Continuously reinforced slabs designed to minimize cracking and curling can probably resist all but the most severe ground movements that could be potentially created by expansive or collapsing soils with minimal distress.

Muller (1984) gives an example of a reinforced concrete canal lining without joints.

It is recommended that the design procedure for concrete-lined channels include the following steps to allow for soil movement and pressure relief:

A. Characterize the soil profile, subgrade stiffness, drainage characteristics and expansion and/or collapse potential of the soils in the geotechnical investigation considering guidelines in Section 4.1.

B. Estimate potential ground movements of moisture sensitive soils using methodology given in Section 4.2.

C. Select the structural and/or subgrade treatment based on an assessment of the calculated risk of moisture changes beneath the slab and the soil structure interaction methodology and possible subgrade treatments given in Sections 4.3 and 4.4.

D. If the subgrade has poor drainage or the water table has the potential for rising above the base of the slab, evaluate the need for pressure relief considering calculated risk and pressure relief design considerations given in Section 4.5.

It is emphasized that ground movements due to expansive or collapsing soils and pressure relief will influence design for a relatively small percentage of projects in the study area. However, when moisture sensitive soil or drainage problems exist, they must be carefully and thoroughly addressed in design on a site specific basis.

4.1 Identification of Moisture Sensitive Foundation Soils

The geotechnical investigations for pavements, roadways and structures conducted in conformance with Arizona Department of Transportation (ADOT) criteria will usually provide adequate site characterization for slab design. Where moisture sensitive subgrades are involved or channels extend well outside the roadway corridors, additional work may be needed.

4.1.1 Collapsing Soils

Collapsing soils can be identified by simple index tests in conjunction with local experience and recognition of landforms by photogeologic methods. As previously stated, the collapsing soils described herein are generally alluvial fan deposits, and are mainly silty sands, clayey sands, and sandy silts. Typical index properties (Beckwith, 1979) are as follows:

Plasticity Index	<10
Dry Density	<95 pcf
Moisture Content	<8 percent, sometimes <4 percent
Standard Penetration Resistance	<15 and sometimes <6 blows per foot

Figure 1 illustrates a widely used empirical identification procedure developed by the Bureau of Reclamation (Gibbs and Bara, 1962) based on dry density and liquid limit. When the moisture content at saturation is above the liquid limit, soils are considered to possess a high tendency toward collapse. Another empirical method (Table 2) presents guidelines developed by Jennings and Knight (1975) based on one-dimensional consolidation tests (ASTM D4186-82). The guidelines are based on the percent collapse under a 4,000 psf pressure. Consolidation tests are usually performed on ring-lined barrel samples (ASTM D3550-84) in local practice. In special cases, samples are obtained by Dennison core sampling and the CME continuous sample tube system, wherein soil sampling is done continuously during auger drilling.

One-dimensional consolidation tests provide the best basis for calculating collapse settlements and are performed using several different procedures. The Jennings-Knight (1959) double oedometer (consolidation) test is a sometimes used. As shown in Figure 2, companion samples are tested at in situ and increased moisture contents. The divergence of the two curves is a manifestation of the shear failure of the bonds between the silt and sand grains. The same curve can be developed by running several single point collapse tests at different pressures (Figure 3). Sometimes slight expansion will occur at low pressures and collapse will take place beyond a certain threshold pressure as shown by Figure 4.

SHB has adopted the philosophy of conducting single five point consolidation tests, as shown in Figure 5, to obtain the essential information for determination of collapse potential. The test is conducted with sample inundation at stresses in the range of those which will exist in the field. Procedures for estimating collapse settlements are given in Section 4.2.1.

4.1.2 Expansive Soils

Identification of expansive soils can be done by simple index tests along with local experience. Methodology based on correlations between percent clay of the soil and plasticity index are widely used for classification of expansive soils. Figure 6 presents relationships for estimating potential expansion of clayey soils in qualitative terms. These relationships developed by SHB represent a modification of the Van Der Merwe approach (1964) which considers geologic origin of the soils and case histories of expansive soil conditions in the Southwest. The work of Seed and others (1962) was heavily considered in developing Figure 6.

One-dimensional consolidation-swell tests are also widely used to evaluate expansive potential.

Procedures for estimating potential expansion of in situ clay soils are outlined in Section 4.2.2.

4.2 Estimating Potential Ground Movements of Moisture Sensitive Soils

The following procedures are recommended to estimate the maximum potential ground movements.

4.2.1 Collapsing Soils

It is recommended that potential ground movements of collapsing soils be computed by conventional methods, wherein the compressibility of soils is determined from strains measured in consolidation tests. In this procedure, settlements at low natural moisture contents are computed on the basis of the difference between strains at the vertical stresses before construction and those involved with addition of embankment and water loads. Sometimes it is necessary to consider stress release due to excavation of overburden in this part of the problem.

Post-construction collapse settlements are then calculated on the basis of the differences in strains between the consolidation curves for natural and saturated conditions. Calculations are made on a one-dimensional basis, even though it is recognized that some degree of three-dimensional movement usually occurs. Experience shows this procedure usually provides a reasonably accurate estimate of settlements.

An alternative approach is to assess the collapsing soils by the empirical methods discussed in Section 4.1.1 and estimate settlements on the basis of performance of other structures on the same or similar deposits.

4.2.2 Expansive Soils

It is recommended that estimation of expansion potential for moderate to highly expansive soil deposits be performed by the modification of the Van Der Merwe method (1964) described below. The method is based on the assumption that initial moisture contents are at the low natural values beneath well drained desert surfaces which are in balance with the semi-arid environment. Experience has shown that this method has been relatively accurate in estimating potential swell for most expansive soils in the Southwest. The total heave beneath a structure is estimated by use of the following simple relationship:

$$H_T = \sum_{d=1}^{n} F_d (PE)_d$$

where H_T = total heave, feet
F_d = reduction factor for depth of soil layer d
PE_d = potential expansiveness of soil layer d, inch/ft.

The potential expansiveness is estimated from the results of Atterberg limits determinations and grain-size distribution of the soil including hydrometer analysis to determine the percent nominal clay sizes. Based on these tests, Figure 6 is used to determine PE.

Table 4 presents values of the reduction factor, F, versus depth. Input of the reduction factor accounts for the decrease in potential heave with depth below the soil surface due to confining pressure. In this procedure, the expansive soils are usually subdivided into layers of no more than 3 feet in thickness with expansion estimated separately for each layer.

4.3 Soil-Slab Interaction Analysis

Several methods have been developed for soil-slab interaction analysis for building slabs bearing on expansive soils (Post-Tensioning Institute, 1980; Mitchell, 1986; Pidgeon, 1980; Wray, 1980; Tucker and Poor, 1977). With some modification, these methods can be used to assess the soil structure interaction for concrete linings on both expansive and collapsing soils. In these methods, the maximum potential subgrade movement caused by wetting of the moisture sensitive soils is estimated by procedures such as those given in Section 4.2. Expansion or collapse inevitably occurs in localized areas. Typically, movements increase from zero to maximum values in 15 to 30 feet. In soil-structure interaction analysis, the shape of the ground movement profile is determined by empirical methods or judgment. The slab is designed to span or cantilever various distances based on this profile which increase with the severity of ground movements. These procedures allow the evaluation as to whether or not a given slab design will perform properly when subjected to the potential ground movements and evaluation of stiffening slabs to resist movements. These methods are expected to show that continuously reinforced slabs designed to minimize cracking and curling will safely resist ground movements for the majority of Class 3, 4 and 5 sites in the study area.

4.4 Treatment of Moisture Sensitive Soils

If potential ground movements are found to be excessive, they can be reduced by stabilizing all or part of the moisture sensitive layer. Another approach is to reduce the probability of excessive movements by providing moisture protection such as geomembrane lining beneath the slab. Positive site drainage which directs runoff away from the edge of the slabs and prevents long-term ponding is another measure to reduce risks. From analysis of calculated risks (Casagrande, 1965), it is sometimes concluded that the risk and consequences of distress are such that treatment is not warranted.

Approaches to the stabilization of moisture sensitive soils are discussed in Sections 4.4.1 and 4.4.2.

4.4.1 Collapsing Soils

Wetting of collapsing soils and vibratory surface compaction has been widely used for Cases 3 and 4 where the thickness is less than about 4 feet. In this procedure, the collapsing soils are wetted by sprinkling or flooding and the surface compacted with about five passes of a heavy steel drum vibratory roller. On large projects, test sections are sometimes done to establish the compactive effort. This procedure has been used successfully on many projects in Arizona, New Mexico and west Texas for which SHB was the geotechnical engineer and in other cases reported in literature (Williams and Marais, 1971; Wolmarons, 1975; Jones and Van Alphen, 1980; Weston, 1980; Schwartz and others, 1981; Barrett and others, 1984). In this procedure, infiltration rates of the wetting front into the soil are typically in the range of 1/2 to 3 feet per day (Bara, 1975; Knodel, 1981; Molle and others, 1981; New Mexico Highway Department, 1982; Shaw and Johnpeer, 1985).

Where collapsing soils are deeper than 4 feet, treatment usually consists of removal to 4 feet from the bottom of the layer, wetting and vibratory compaction, then bringing the area up to grade with compacted fill. Studies by the New Mexico Highway Department (1982) confirm work by SHB which indicates this approach is usually more economical than deep dynamic compaction or vibroflotation.

Sometimes the entire collapsing layer is removed and replaced with structural fill to provide a more positive treatment and a more simple construction procedure.

4.4.2 Expansive Soils

Overexcavation of the expansive soils and replacement with nonexpansive structural fill to the depth necessary to limit potential movements to tolerable limits is the most common treatment of expansive soils. Placing a geomembrane lining beneath and adjacent to the slab can be done to minimize seepage into the expansive soils. The geomembrane can be extended below grade in vertical trenches near the edge of the concrete lining as described by Steinberg (1980) to minimize lateral migration of water. Membrane protection has long been used by the Bureau of Reclamation (Holtz, 1959).

4.5 Drainage & Pressure Relief

A significant rate of leakage usually occurs through concrete lined canals and channels. Table 5 shows canal seepage data reported by Wilson (1980) and Halderman (1980). Measured seepage losses from Salt River Project canals as reported by Wilson (1980) varied from 0.0118 $ft^3/ft^2/day$ for concrete canals in good condition to 0.2363 $ft^3/ft^2/day$ for concrete canals in fair condition. The data in Table 5 applies to thin, lightly reinforced or unreinforced linings. Continuously reinforced linings and thicker, plain concrete linings with carefully maintained joints would probably experience less seepage.

Where the subgrade is not free draining and rapid drawdown of water levels occurs, "blow outs" and serious cracking and distortion of concrete linings sometimes occurs. Damage due to hydrostatic pressure buildup has been experienced for concrete slabs for drainage channels, containment basins at water and sewage treatment plants, canals, swimming pools and tailings thickeners at mineral processing plants.

No drainage and pressure relief is necessary for Cases 1 and 4 where free draining subgrades are present beneath the critical parts of the lining. The decision whether or not to provide pressure relief for Cases 2, 3 and 5 which involve poorly draining subgrades should be made considering the depth of the channel, nature of lining, probability of significant water accumulating under the lining and consequences of failure. For example, shallow canals with continuously reinforced lining would have a relatively low susceptibility to damage. Cases where water will be present in the channel for long periods of time or where external sources of water such as heavy landscaping are present adjacent to the channel would have a higher risk of problems.

Drainage and pressure relief should also be provided in cases where the water table could rise above the bottom of the concrete lining after construction.

Where pressure relief is needed, the system should consist of a grid of geotextile or geocomposite drains leading to flap-type valves at the bottom of the channel. Methodology by Koerner (1986a, 1986b) should be used in drainage system design.

The extent or spacing of geocomposite drainage strips under the concrete lining and the appropriate type of geocomposite should be determined for each application. The potential flow rates required for drainage should be initially estimated. The geocomposite utilized should be capable of transmitting the design flow rate at the particular hydraulic gradient and confining stress. Geocomposites such as Miradrain 4000, Enkadrain, Ameridrain II, and Filtram are typically used for medium to high required flow rate (>1 gal/min-ft) applications.

Flow capacities of these geocomposites are presented by Koerner (1986a, 1986b).

4.6 Geotechnical Engineering Design Parameters

Recommended maximum cut slopes, moduli of subgrade reaction for the design of slabs for wheel loads and lateral earth pressures for design of retaining walls are presented in Table 3.

The recommended maximum slopes are those that could be safely used from a stability standpoint. In most cases, slopes are controlled by construction considerations for the lining and do not exceed 1.5:1 (U.S. Department of Interior, 1978; Pavel, 1982).

Moduli of subgrade reaction (k) for "wet" conditions for Cases 3, 4 and 5 apply to situations where the moisture sensitive soils are not stabilized or replaced with structural fill. Where stabilization or replacement is done, the k-values for dry conditions should be used. Often for Cases 3, 4 and 5, the moisture sensitive soils will underlie the upper parts of the slopes and sands and gravels or cemented soils will underlie the channel bottom. In these situations, k-values for Cases 1 or 2 should be used for analysis of wheel loads on the bottom slab. Extensive plate bearing tests on local soils reported by Crossley and Beckwith (1978) were considered in developing recommended k-values.

The design lateral earth pressure for retaining walls will be dependent upon the rigidity of the wall system and the slope of backfill behind the wall. Where walls are free to move at the top, active earth pressures will be developed and, where essentially rigid, reinforced concrete walls are restrained from movement at the top, at rest earth pressures will be applicable to design. A summary of both active and at rest lateral earth pressures for the conditions of horizontal backfill and for backfill sloping at 10, 20 and 30 degrees from the horizontal are presented in Table 3. The earth pressures are applicable only when free drainage of the backfill material behind the wall is provided by use of weep holes, gravel drains or pipe drains. Valves would be needed at weep holes for channel applications.

4.7 Concrete

Recommended guidelines for concrete mix designs are summarized in Table 6. The guidelines are intended to achieve as low drying shrinkage as practicable considering constructibility and traffic requirements on the lining. The shrinkage tests on concrete panels in accordance with procedures suggested by Kraai (1985) are believed to be a critical part of the mix design process to minimize shrinkage.

Higher early strength for given mix design criteria will result in higher drying shrinkage, so the trade off between early traffic and higher shrinkage must be considered in design.

The minimum slab thickness of 6 inches for continuously reinforced slabs, dictated by construction considerations, will be sufficient to support design wheel loads for most sites. However, for some soft subgrades, thickness design for wheel loads may confront the design engineer with the choice of specifying higher strength, higher shrinkage concrete to achieve a 6-inch thickness, or using a greater slab thickness of minimum shrinkage concrete.

Ytterberg (1987a) recommends coarse, well graded aggregate up to one-third the slab thickness to minimize cement content and shrinkage. However, based on local experience, well graded aggregate up to 1 inch maximum size with fly ash mineral filler is recommended. It is believed this will achieve better constructibility and a very low shrinkage mix.

In the initial part of implementation of these guidelines, a series of mix designs and field shrinkage tests should be performed in accordance with the guidelines in Table 6 to establish the reference mix design which will provide the criteria for acceptance of project mixes.

Salt River aggregate, locally produced Type II cement and appropriate water reducers should be used. This series of trial mixes should lead to a reference mix design and control shrinkage test panels which represent the lowest practicable drying shrinkage that can be obtained with local materials.

4.7.1 Aggregates

Salt River aggregates or an approved equivalent should be specified. Composition of coarse and fine mineral aggregate fractions should be optimized to produce a maximum density grading with low voids content. This evaluation should be accomplished by use of the 0.45 power gradation chart plot of the aggregate source composite gradation results. Aggregates should meet requirements of ASTM C33 with cleanliness of the fine aggregate fraction at a maximum of 3 percent passing the no. 200 sieve by elutriation. To accomplish a well graded coarse aggregate fraction (plus no. 4), it may be necessary to blend a coarse ASTM no. 57 with a no. 7 or no. 8 maximum size coarse aggregate with 100 percent passing 1-inch sieve.

4.7.2 Fly Ash Mineral Filler

Locally available fly ash (ASTM C618, Class F or C) should be added as a silt mineral filler to the composite grading of coarse and fine aggregate to accomplish a fraction passing the no. 200 sieve in the range of 8 to 10 percent. This will be about 6 percent fly ash mineral filler by weight of coarse and fine aggregate. Local aggregate production will generally produce a composite in the range of 57 percent of coarse fraction, 37 percent fine fraction and 6 percent fly ash mineral filler. The filler-aggregate blend will generally increase concrete unit weight on the order of 2 pcf, increase compressive strength for a given cement factor, reducing permeability, drying shrinkage and cracking.

4.7.3 Chemical Admixtures

The use of water reducing agents is essential to achieve efficient mix designs for the hot, dry climate of the Phoenix area.

Ytterberg (1987a) states that high range water reducers sometimes increase rather than decrease drying shrinkage. Thus, an important reason for performing shrinkage tests on concrete panels (Kraai, 1987) in the field is to confirm that proposed admixtures will not significantly affect drying shrinkage in relation to the control test panels.

4.7.4 Cement

Only Type II cement meeting the requirements of ASTM C150 should be used. Where cements other than those previously approved on the basis of field shrinkage tests are proposed for use, mortar shrinkage tests in accordance with ASTM C157 should be run as the first step in the evaluation process. Cements with shrinkage greater than the approved cements should be rejected.

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TABLE 1

RECOMMENDED SUBGRADE TREATMENT, DRAINAGE & PRESSURE RELIEF PROCEDURES FOR THE FIVE TYPICAL SUBSURFACE PROFILES IN THE GREATER PHOENIX AREA

Subsurface Profile Case	Description	Subgrade Treatment	Drainage & Pressure Relief
1	Clean sands or sands & gravels	° No special treatment required ° Scarification & recompaction of surface soils	° Positive site grading away from canal side slopes ° Pressure relief not required unless potential exists for groundwater to rise above canal bottom
2	Cemented desert alluvium	° No special treatment required ° Scarification & recompaction of surface soils	° Positive site grading away from canal ° Low risk of water accumulation: Pressure relief not required ° High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise Geocomposite drainage strips with pressure relief valves
3	Moisture sensitive soils over poorly drained cemented desert alluvium	° Collapsing soils ° < 4-feet thick: partial over-excavation, wetting, vibratory compaction & replacement with compacted fill ° Full removal & replacement with compacted fill ° Expansive Soils ° Partial or total removal & replacement with compacted fill ° Geomembrane underliner as seepage barrier	° Same as Case 2

TABLE 1 (con'd.)

RECOMMENDED SUBGRADE TREATMENT, DRAINAGE & PRESSURE RELIEF PROCEDURES
FOR THE FIVE TYPICAL SUBSURFACE PROFILES IN THE GREATER PHOENIX AREA

Subsurface Profile Case	Description	Subgrade Treatment	Drainage & Pressure Relief
4	Moisture sensitive soils over free draining granular strata	° Same as Case 3	° Same as Case 1
5	Expansive clays throughout profile	° Overexcavate & replace with nonexpansive compacted fill; depth of overexcavation as required to limit potential expansion to tolerable limits ° Geomembrane underliner as seepage barrier	° Same as Case 2

TABLE 2

RELATIONSHIP BETWEEN ESTIMATED
COLLAPSE POTENTIAL AND PROBLEM SEVERITY

Collapse Potential*	Severity of Problem
0 to 1%	No problem
1 to 5%	Moderate Trouble
5 to 10%	Trouble
10 to 20%	Severe Trouble
>20%	Very Severe Trouble

*Under a 4,000 psf surcharge load (after Jennings and Knight, 1975)

TABLE 3

RECOMMENDED ENGINEERING DESIGN PARAMETERS FOR SUBSURFACE CONDITIONS 1 THROUGH 5

Case	Subsurface Conditions	*Slopes Cut	*Slopes Fill	Modulus of Subgrade Reaction, pci Dry	Modulus of Subgrade Reaction, pci Wet	Lateral Earth Pressures Against Retaining Walls "Active" β, deg. 0	10	20	30	"At Rest" β, deg. 0	10	20	30
1	Clean sand or sand & gravel	2:1	2:1	600	600	30	31	37	56	50	52	61	93
2	Moderately to strongly cemented alluvial soils	1:1	1:1	750	600	30	31	37	56	50	52	61	93
3	Moisture sensitive (collapsing or expansive) soils over cemented alluvium	1:1	1:1	200	100	30	31	37	56	50	52	61	93
4	Moisture sensitive (collapsing or expansive) soils over granular free-draining soils	2:1	2:1	200	100	30	31	37	56	50	52	61	93
5	Medium to highly expansive clays throughout entire profile	1:1	1:1	600	400	30	31	37	56	50	52	61	93

*Notes:

1. Recommended slope ratios are horizontal to vertical. Slopes are maximum safe slopes. In most cases, slopes will be controlled by construction considerations and will be no steeper than 1.5:1.

2. Moduli of subgrade reaction for Cases 3, 4 and 5 for wet conditions are based on the moisture sensitive soils not being stabilized or replaced with structural fill. Values for dry conditions for these cases apply to stabilized moisture sensitive soils or structural fills.

3. "Active" case for lateral earth pressures applies to conditions in which the retaining wall is free to move at the top. The "at rest" case applies where walls are restrained from movement at the top. The angle β refers to the slope angle of the backfill from the horizontal.

TABLE 4

VALUES OF FACTOR, F, VERSUS DEPTH, d

d=20 log F

Depth Feet	Factor F
0 - 1	0.943
1 - 2	0.842
2 - 3	0.750
3 - 4	0.668
4 - 5	0.596
5 - 6	0.531
6 - 7	0.473
7 - 8	0.422
8 - 9	0.376
9 - 10	0.335
10 - 11	0.298
11 - 12	0.266
12 - 13	0.237
13 - 14	0.211
14 - 15	0.188
15 - 16	0.168
16 - 17	0.150
17 - 18	0.133
18 - 19	0.119
19 - 20	0.106
20 - 21	0.094
21 - 22	0.084
22 - 23	0.075
23 - 24	0.067
24 - 25	0.060
25 - 26	0.053
26 - 27	0.047
27 - 28	0.042
28 - 29	0.038
29 - 30	0.034

TABLE 5

SEEPAGE LOSSES FOR CONCRETE LINED CANALS
AFTER WILSON (1980), HALDERMAN (1980)

Seepage Losses $Ft^3/Ft^2/day$	Explanation
0.03 - 0.059	Average water depth 3 - 3.5 ft. (average for 5 ponding tests)
0.04 - 0.223	Average water depth 2.5 - 4 ft. (average for 5 ponding tests)
0.231 - 0.374	3 tests, average depth below 1 ft; 2 inch concrete
0.03 - 0.16	Measured on concrete-lined canals, Lower Rhone-Languedoc irrigation scheme, France.
0.07	3-1/2 inch concrete; Friant Kern Canal, Central Valley, Calif., water depth 17.2 ft.
0.53	4 inch thick concrete, seepage measured by inflow-outflow.
0.008	Reach 1. Contra Costa Canal, Central Valley Project, Calif., design discharge 140 cfs; reinforced concrete; design depth approx. 5 ft.
0.09	Reach 2. Contra Costa Canal, Central Valley Project, Calif.; design discharge 140 cfs; reinforced concrete; design depth approx. 5 ft.
0.0110	SRP Canal; concrete in good conditon
0.2363	SRP Canal; concrete in fair conditon

TABLE 6

Recommended Guidelines for Concrete Design

Mix & Evaluation of Drying Shrinkage

DESIGN MIX

- Design mix should meet the general specification requirements of ADOT 1006-3.

Strength

- Compressive strength should be 3,000 psi at 28 days.

Aggregates

- Aggregates should meet minimum requirements of ADOT Standard Specification 1006-3. Coarse aggregate should be size 57. Coarse aggregate should have a minimum of 75 percent crushed faces.

Mineral Filler

- Fly ash Class F (ASTM C618) should be used as a mineral filler. Loss on ignition should be a maximum of 3.0 percent. Fly ash should not be considered as a replacement for cement. Fly ash should have an R factor less than 2.5. The R factor is defined as (C-5%)/F, where C is the calcium oxide content expressed as a percentage and F is the ferric oxide content expressed as a percentage. The R factor requirement may be waived if the contractor furnishes documented test results that the soil in contact with the Portland Cement concrete contains less than 0.10 percent water soluble sulfate, (as SO4) and/or the water in contact with the Portland Cement concrete contains less than 150 milligrams per liter sulfate (as SO4). The tests for sulfates should be performed in accordance with the requirements of California Department of Transportation Test Method No. 417. Calcium and ferric oxide content should be determined in accordance with the requirements of ASTM C311.

TABLE 6 (CONT'D.)

Recommended Guidelines for Concrete Design

Mix & Evaluation of Drying Shrinkage

Chemical Admixtures

° Should meet the requirements of ADOT 1006-2.04.

Water

° Should meet the requirements of ADOT 1006-2.02.

Cement

° Should be Portland Cement Type II, meeting the requirements of ASTM C150.

Slump

° Maximum 4 inches (AASHTO T119).

Air Content

° 5 plus or minus 2 percent by volume (AASHTO T-152).

Curing

° Should meet the requirements of ACI 301, Chapter 12.

° Subgrade should not be wetted prior to placement of concrete.

Hot Weather Concreting

° Should meet the requirements of ADOT 1006-5.02

Minimum Cement Content

° Not applicable.

TABLE 6 (CONT'D.)

Recommended Guidelines for Concrete Design

Mix & Evaluation of Drying Shrinkage

DRYING SHRINKAGE EVALUATION

Mortar Shrinkage Tests

° ASTM C157, "Length Change of Hardened Cement Mortar and Concrete," testing should be performed on the cement proposed for the project design concrete mix. If other than previously approved Type II cement is proposed, the shrinkage of the cement should be equal to or less than the value obtained in the control specimens made from previously approved cements which result in the lowest practicable shrinkage.

Field Shrinkage Tests

° Test panels should be prepared with the proposed concrete design mix for purpose of evaluating drying shrinkage properties. Test panels should be made in accordance with the Kraai Method outlined in Concrete Construction, Volume 30, No. 9, September, 1985, 9 pp. 775-788). Test panels shall be 2 by 3 feet in plan dimension and 2 inches in thickness.

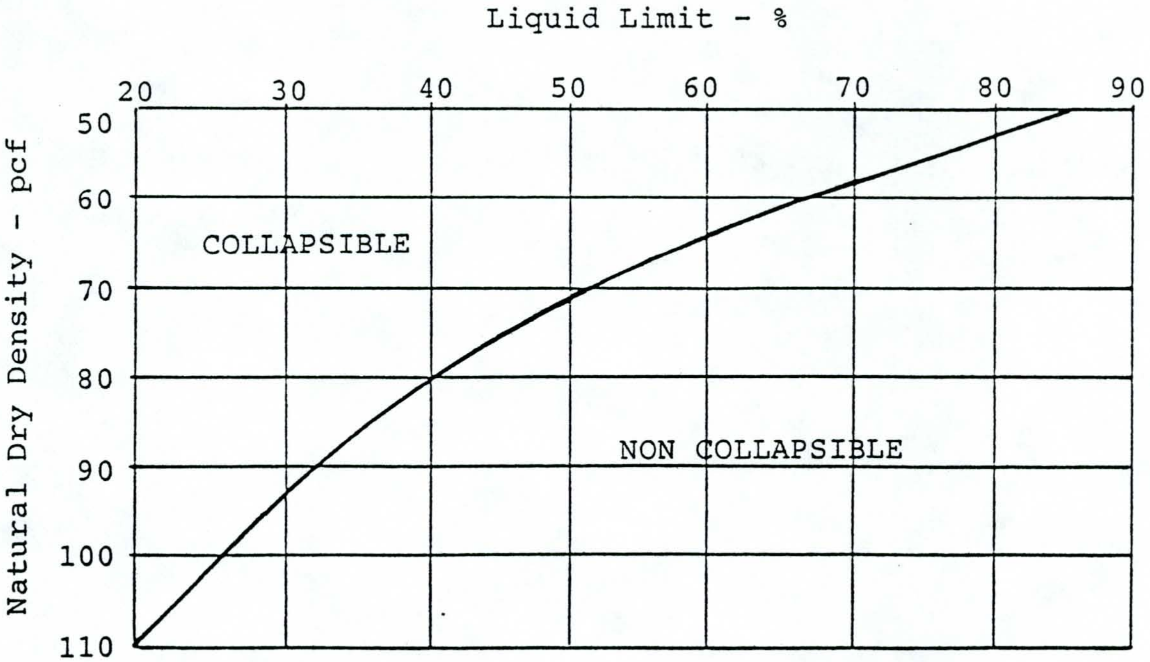
° The Control Test Panel should be made from an established reference mix design. Locally produced Salt River aggregate should be used. Minimum compressive strength should be 3,000 psi at 28 days. Fly ash, as a pozzolanic material, should be utilized as a mineral filler at a maximum of 90 pounds per cubic yard of concrete. A water reducing admixture should be used meeting the requirements of ADOT 1006-2.04.

The project design mix acceptance should have an equal or reduced number and size of shrinkage cracking as compared to the Control Mix Test Panel.

FIGURE 1

CRITERION FOR COLLAPSE POTENTIAL

AFTER BARA & GIBBS (1962)



Design Guidelines for
Concrete Lined Drainage
Channels
Arizona Department of
Transportation
Phoenix, Arizona
SHB Job No. E87-112

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FIGURE 2

Jennings-Knight Double Oedometer Test

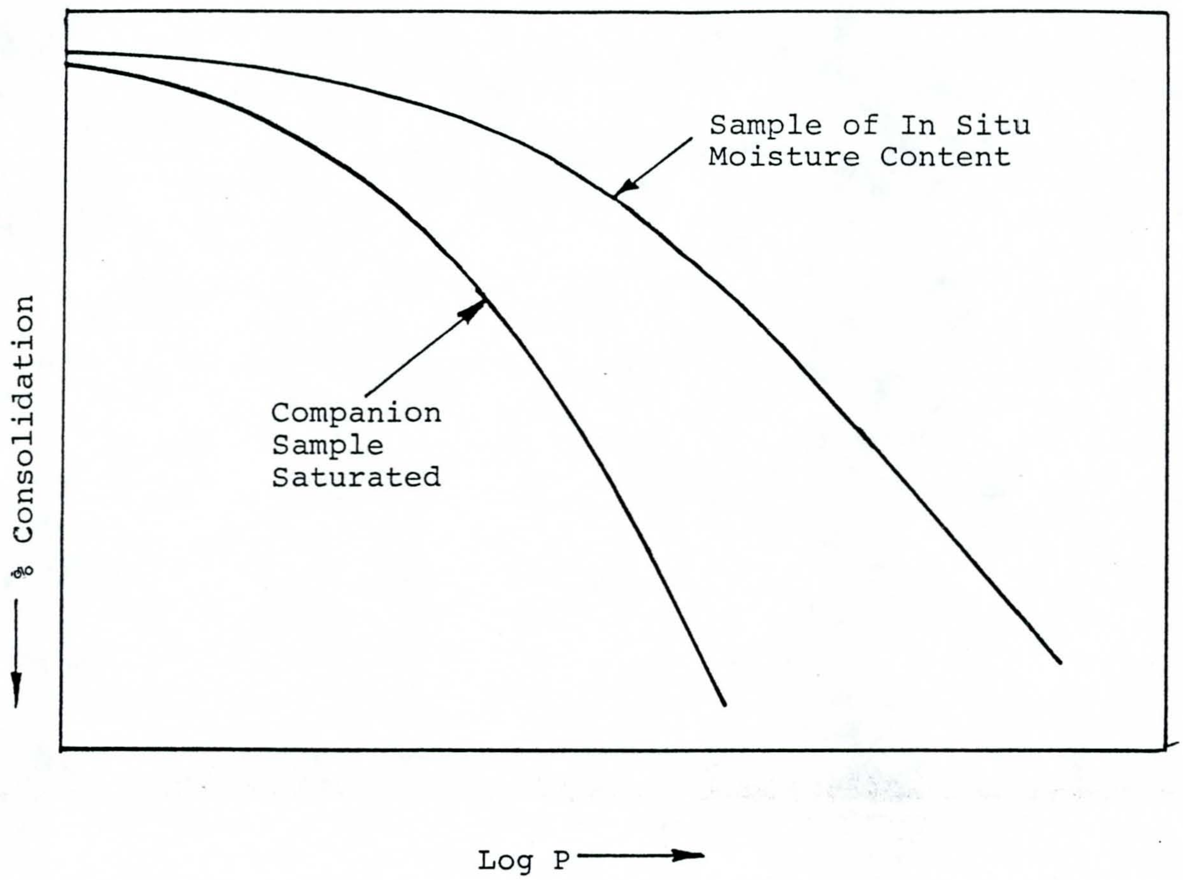

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FIGURE 3

Double Oedometer Developed by
Several One-Point Consolidation Tests

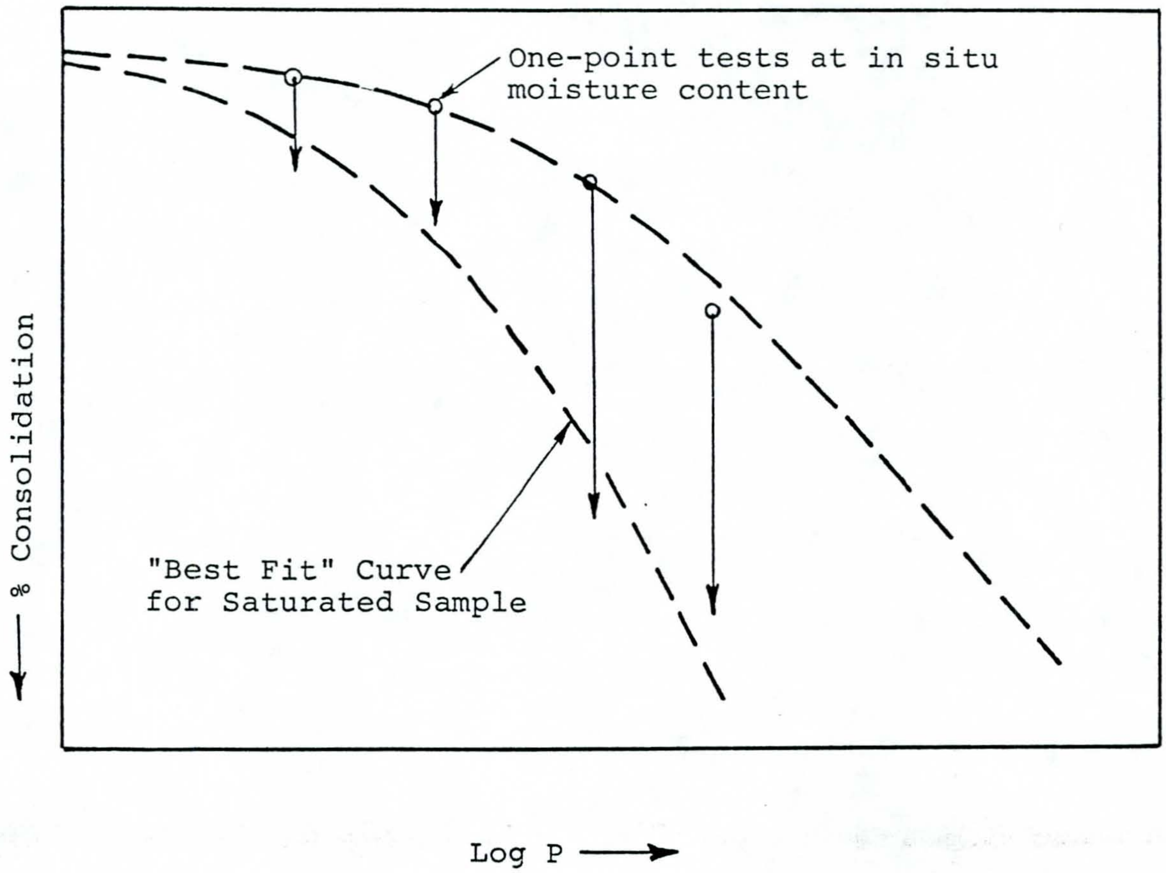
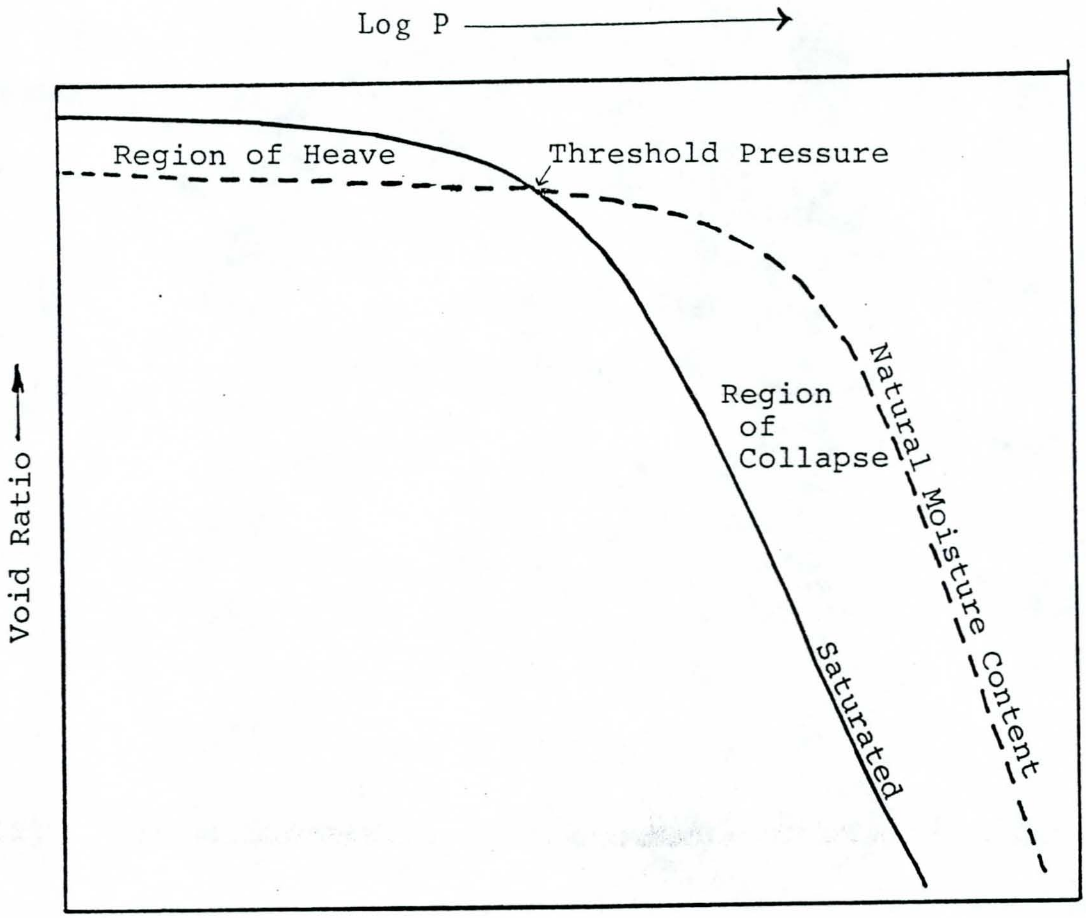


FIGURE 4

Double Oedometer Test Showing
Heave at Low Stresses



Design Guidelines for
Concrete Lined Drainage
Channels
Arizona Department of
Transportation
Phoenix, Arizona
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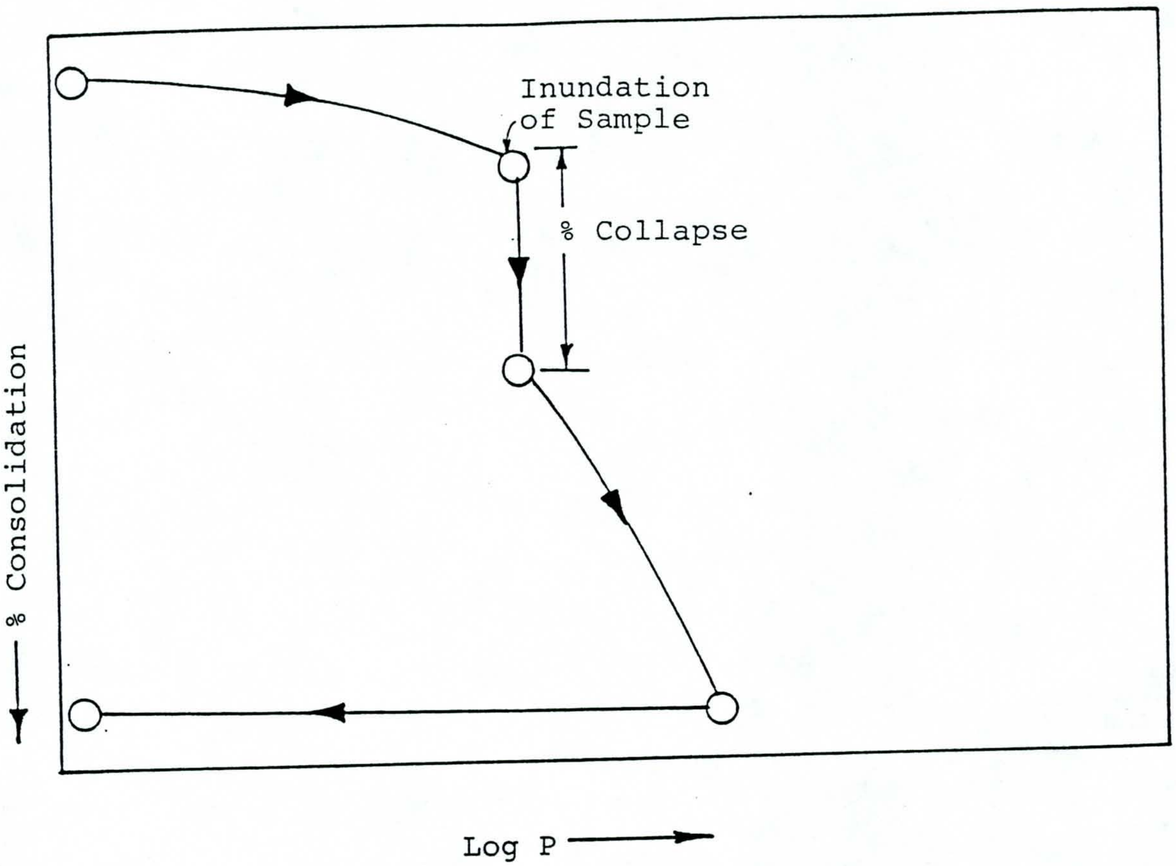
FIGURE 5

<u>Typical Consolidation Test on "Collapsing" Soil</u>



Design Guidelines for Concrete Lined Drainage Channels
Arizona Department of Transportation
Phoenix, Arizona
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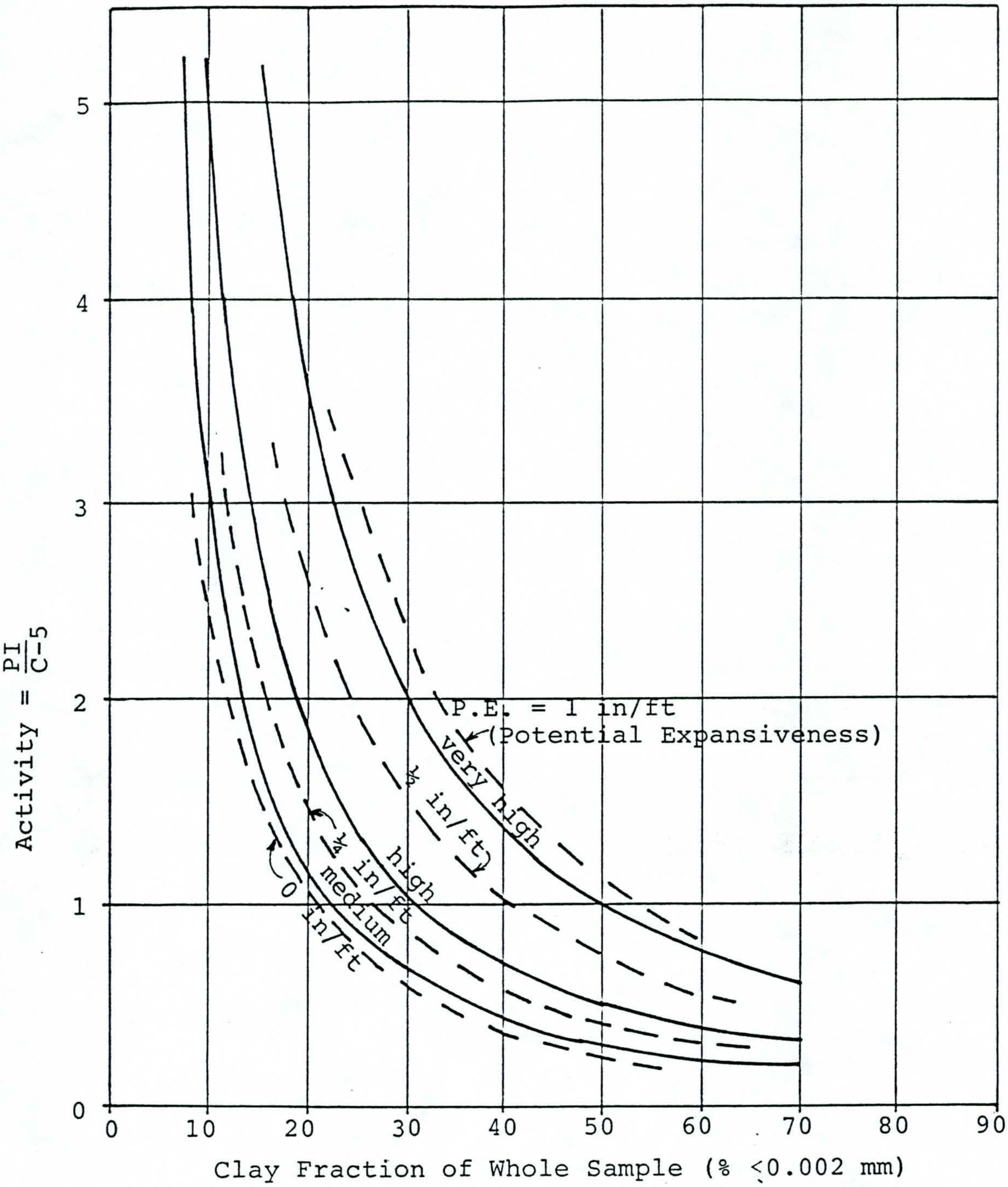
FIGURE 6

Determinations of Potential Expansiveness of Soils



Design Guidelines for
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Channels
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COLLECTED DATA

U.S. BUREAU OF
RECLAMATION

BUREAU OF RECLAMATION PRACTICES FOR DESIGN AND CONSTRUCTION OF CONCRETE LINED CANALS

By Forrest Ritchey, F. ASCE 1/
John G. Starbuck, M. ASCE 2/

Water losses in canals through seepage are reduced through the installation of portland cement concrete lining. Except in specific instances where canal safety is imperative and seepage must be reduced to near zero, the lining is not reinforced because of the high cost. Also, concrete linings eliminate weed growth, reduce flow resistance and maintenance costs, and are impervious to burrowing animals.

To determine the need for canal lining, consideration must be given to seepage rates, value of water saved, operation and maintenance costs, drainage costs or value of land taken out of cultivation by seepage if lining is not provided, canal size, reservoir size, right-of-way, allowable velocities, structure costs, and environmental factors.

Velocity:

The Manning formula is used by the Bureau of Reclamation for computations for open channel flow. The formula is as follows:

$$V = \frac{1.486}{n} r^{2/3} s^{1/2}$$

V = velocity of water in feet per second

S = slope of energy gradient in feet per foot

r = hydraulic radius (water area divided by wetted perimeter)

n = coefficient of roughness

The coefficient of roughness varies with the size of the canal and is shown on figure 1.

Usually velocities should be less than 8 feet per second to avoid the possibility of converting velocity head through a crack to pressure head under the lining and lifting the lining. A design check using an "n" value of 0.003 less than the design "n" used for the lining is also required to make certain the depth of flow does not approach critical depth closely enough to develop standing waves at sections where the bottom might be raised above theoretical grade due to construction tolerances. The alinement for a concrete-lined canal should have a minimum radius of three times the water surface width.

1/ Supervisory Civil Engineer, Head, Canal Structures and Bridge Section, Bureau of Reclamation, Denver, Colorado
2/ Civil Engineer, Bureau of Reclamation, Denver, Colorado

Hydraulic Properties:

From experience, the steepest satisfactory side slope for a large lined canal from both construction and maintenance considerations is 1-1/2:1. Concrete-lined canals have a ratio of base width to water (B/D) depth of from 1 to 2. Small canals normally have a ratio of nearly 1, while large canals may have a ratio up to 2. The section with the least wetted perimeter is the most economical. Figures 2 and 3 show Bureau of Reclamation standard dimensions and hydraulic properties for small canals with concrete lining. Figure 4 shows normal lining thickness.

Foundation:

Concrete lining requires a firm, dense foundation of existing or compacted earth with a smooth surface to receive the lining. In low density areas the canal is overexcavated and backfilled with suitable compacted earth. When partial backfilling of an existing canal is necessary to reduce the cross-sectional area to that required for a lined canal, the backfill must be compacted to such a degree that subsequent settlement will not rupture the lining. In rock excavation except shale, the portion of the section to be covered with concrete lining is overexcavated so that there will be not less than 3 inches between rock points and the underside of the lining. Surfaces so excavated are filled with compacted pervious material. Where expansive clay or shale is encountered the canal prism is overexcavated a minimum of 2 feet below the underside of the lining, measured normal to the bottom and side slopes. The excess excavation is refilled to the underside of the concrete lining with selected compacted impervious material. Where lens or lenses of shale occur in the foundation the entire lining foundation is overexcavated above the bottom of the lowest shale lens 2 feet below the underside of the canal lining. The excess excavation is refilled to the underside of the concrete lining with selected compacted impervious material. The required degree of compaction varies with the soil, in-place materials, thickness of the backfill, and type of lining to be used. These related factors must be given special consideration for each such installation.

Concrete Lining:

Concrete canal lining is subject to complex stresses resulting from temperatures or moisture change in the slab or from a combination of the two.

The compressive stress resulting from either a temperature or moisture increase is of little concern for two reasons. First, a slab which is fully restrained at both ends and subjected to a 100° F increase in temperature will develop only about 1,500 pounds per square inch of compressive stress (assuming 0.000005 for the coefficient of expansion and 3,000,000 pounds per square inch for the modulus of elasticity in the relation E = stress over strain). This is considerably below the average compressive strength of good concrete. Secondly, the expansion of concrete due even to complete saturation is never as great as the contraction that results from the hardening and drying of the concrete shortly after placing. Contraction cracking is of primary concern in concrete lining, but cracking can be controlled by the forming of contraction joints at proper intervals. Where lining operations are continuous, a weakened-plane type contraction joint formed with special plastic water stops or "sidewalk" groove is formed in the concrete to a depth of about one-third of the lining thickness to induce cracking at these predetermined planes of weakness. The grooves are filled with polysulfide to prevent leakage.

Portland cement concrete for canal lining may be economically placed by a variety of methods. Jobs involving considerable lengths of large canals usually utilize longitudinally operated slip-forms supported on rails placed along both berms of the canal or track-laying machines. Short lengths of large canals often do not justify the use of slip-form machines; these may be lined by use of winch-drawn screeds operating transversely up each side slope.

Concrete Mix:

Details of concrete mix design and recommended practices for both large and small canals are covered in the Bureau's Concrete Manual [5].3/ Concrete for lining a canal should be plastic enough for thorough consolidation and stiff enough to stay in place on the side slopes. Usually a slightly oversanded mix is needed for machine-placed lining to give adequate workability. Maximum aggregate size is 1-1/2 inches. Close control of the workability and consistency of the concrete is important, because a variation of 1 inch in slump can seriously interfere with the progress and quality of the work. The use of an air-entraining agent in the mix is recommended and is particularly important where exposure to freezing temperatures is anticipated. Other admixtures such as set retarders are also used but have not proved fully satisfactory.

Placing Concrete:

The large slip-form machines supported on railroad rails have been in general use for about 30 years. The equipment is highly mechanized and consists essentially of a framework which travels on rails (one on each bank of the canal) supporting a working platform, either distributor plates or drop chutes, a compartmented supply trough, a vibrator tube in the bottom of the trough, and the slip-form. The latter is a

3/ [redacted]rs i[redacted]cket[redacted]icat[redacted]eren[redacted]mber[redacted]

steel plate curved up at the leading edge, extending across the bottom and up the slopes of the canal, which forms and smoothes the finished surface of the lining. If a distributor plate is used, it is fastened to the leading edge of the slip-form and extends upward on a steep incline to the working platform at the top of the machine. The concrete is dumped onto the distributor plate which serves to spread the concrete as it is deposited directly ahead of the slip-form. If drop chutes are used, these supply concrete from a row of hoppers in the working platform to compartments in the trough below. Concrete is usually dumped into the working platform hoppers or onto the distributor plate from a shuttle car loaded by bucket or conveyor belt from mixers on the bank. As the concrete passes out at the bottom of the trough and under the slip-form, it is consolidated by a vibrating tube, having a minimum frequency of 4,000 cycles per minute, mounted parallel to and a few inches ahead of the leading edge of the slip-form. The metal slip-form is equipped with form-type vibrators.

Concrete for these large slip-forms is usually mixed at a central plant although on some jobs the slip-forms are supplied by transmit-mix trucks. The rate of lining placement in large canals requiring 10 to 12 square yards of lining per linear foot of canal often exceeds 1,000 linear feet per day.

The track-laying machine, which is supported and driven by crawler-type tracks, is very similar in detail to the rail type previously described except for the method of propulsion. It is hydraulically operated and guided by a sensor device with feelers which are in contact with a wire or string line. Many trimmers are guided by the wire line and the liner by skis riding on the trimmed subgrade. This reduces concrete overrun. With experienced operators and close inspection of the work, the machine is capable of placing an acceptable lining.

Concrete Finish:

If the water conveyed in the canal is relatively clear and if experience in the locality indicates that little moss or algae growth on the lining can be anticipated, the original surface finish will probably be effective throughout most of the life of the lining. In this case, a smooth finished surface which would increase the carrying capacity of the canal may be warranted. However, if the water will carry considerable sand or silt which may be deposited in the canal or if the surface of the lining may become covered with moss or algae growth, either of these two conditions may have a greater effect on the efficiency of the canal than the degree of original surface finish; in these circumstances a very smooth, hand-troweled surface would be of little value and the cost of securing it would be unjustifiable. Since a majority of irrigation canals carry water which contains a certain amount of sand or silt and many are subject to the growth of moss or algae, a reasonably smooth surface without voids should be adequate for a concrete lining.

Curing Concrete

The proper curing of portland cement concrete in canal lining is equally as important as the curing in any thin structural section. Moist curing is, in general, preferred, but the use of accepted sealing compounds has been found satisfactory. To assure that the lining does not dry out rapidly, the subgrade should be well moistened immediately prior to placing of the concrete.

A more detailed discussion of both finishing and curing of concrete is contained in the Bureau's Concrete Manual [5].

Expansion joints are not ordinarily required in a concrete lining, except where fixed structures intersect the canal.

Construction Joints:

Construction joints are necessary where lining operations are discontinued at the end of the day or for other reasons and are resumed after a considerable time interval. In Bureau work a construction joint is a properly prepared joint where the previously placed concrete and the fresh concrete are bonded. A contraction joint is a butt joint in which the previously placed concrete is painted with sealing compound to assure that no bonding takes place. Contraction joints should be filled with an elastic material.

The need for better joint and crack fillers for concrete lining has become apparent from field examinations of existing installations.

Grooves:

Longitudinal grooves are advisable in concrete canals having lined perimeters of 30 feet or more. The recommended spacing for both transverse and longitudinal contraction joints in unreinforced concrete varies from 10 to 15 feet, depending on the size of canal and the thickness of lining. Figure 5 shows the recommended spacing and groove dimensions. A more detailed discussion of the spacing of grooves and the methods of forming them is contained in the Concrete Manual [5] under the discussion of contraction joints. Typical plastic waterstops are shown in figure 6.

Freeboard:

Freeboard for lined canals depends upon a number of factors, such as the size of canal, velocity of water, curvature of alinement, storm water entering the canal, wind and wave action, and anticipated method of operation. The normal freeboard varies from 6 inches for small laterals to 2 feet or more for large canals. Figure 7 represents average Bureau practice as a guide for determining minimum freeboard and bank height for canals with hard surface, buried membrane, and earth linings. The height of canal bank above the top of the lining

usually varies from 6 inches for small laterals to over 2 feet for large canals.

Berms:

A 2- to 6-foot berm is normally provided at the top of concrete linings for the construction convenience of trimming and lining machinery. Backfill should be placed on this berm from the top of the lining and sloping upward to the earth bank to prevent surface drainage from entering the subgrade behind the canal lining.

Folsom South Canal shown on figures 8 and 9 is a typical canal constructed by the Bureau of Reclamation. The loose embankments placed over the outside of the compacted embankment provide for operating roads and additional stability. Unsuitable material should be stripped from under uncompacted embankments. The entire surface of the subgrade for compacted embankment should be plowed or scarified thoroughly to a depth of not less than 6 inches, moistened, and compacted. The compacted embankment should be placed in layers not more than 6 inches thick after compaction. The dry density of the soil fraction in the compacted material should not be less than 95 percent of the laboratory maximum density as determined by the Proctor method. The material, when distributed and compacted, should be homogeneous and free from lenses and pockets. The top width of the compacted embankment varies with size and location of canal, type of lining, and other pertinent factors but is usually 2 to 4 feet for canals having a maximum capacity of 100 second-feet and 6 to 8 feet for larger canals. The outside slope of the compacted embankment is normally specified as 1:1.

Line and Grade:

Specifications requirements with respect to line and grade should be as liberal as compatible with good engineering for operating conditions of the canal. Current Bureau specifications for portland cement concrete linings permit departure from established line of 4 inches on curves and 2 inches on tangents and 1 inch from established grade. These generous tolerances permit the effective use of both rail-guided and subgrade-guided lining machines, which in turn has been instrumental in obtaining lower cost linings. Abrupt departure from and return to alinement and grade are not permitted.

Seepage:

Since most canal linings are installed to prevent seepage, the subgrade is usually relatively free draining and above ground-water level. However, when the canal is empty or when the water level in the canal is relatively low, a high ground water will result in unbalanced hydrostatic back pressures on the lining which may be sufficient to damage the lining by flotation, unless it is protected by underdrains.

A similar situation may occur in areas where the canal is lined for reasons other than to prevent seepage and where the impervious strata prevent the free drainage of canal leakage. The accumulation of the water in the soil surrounding the canal may result in a local high ground-water table which, during a period of rapid drawdown of the water level in the canal, may produce damaging hydrostatic back pressures. In regions subject to freezing temperatures, the canal lining may also be severely damaged by the freezing and resultant heaving of the saturated subgrade.

The location of the canal bottom, with respect to the ground-water table, is especially important. In cold climates, the canal bottom must be at least 3 feet above the water table to prevent heaving from freezing and thawing. The greatest danger exists in silts or other highly frost-susceptible soils, especially in areas of frequent cycles of freezing and thawing.

In all instances the probability of damaging the lining can be greatly reduced by providing underdrains. There are two common types of drainage installations. One type for normal ground-water conditions consists of 4- or 6-inch tile placed in graded sand and gravel filters along one or both toes of the inside slopes. These longitudinal drains are either connected to closed outlet pipes which discharge the water outside of the canal into an open drain constructed at an elevation below the canal invert or to a pump pit. Also, flap valve weeps are used to release pressure through the lining. The second type is used for a perched water table and consists of permeable finger drains of selected sand and gravel drained into the canal at 7.5- to 15.0-foot intervals by flap valves in the invert. A drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade is shown in figure 10. Both the tile pipe system and the unconnected flap-valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve.

In areas where freezing can occur, the underdrainage system must be designed to be effective in cold weather if the condition of excessive external pressure can develop during the winter season.

Seepage losses from properly constructed concrete-lined canals will normally be less than 0.07 cubic foot per square foot per day. Technical Bulletin No. 1203 [4] will be helpful in estimating seepage losses in earth.

Safety Features:

Safety ladders are installed on the sides of concrete-lined canals where the vertical lining height is 2.5 feet or more. The ladders are spaced at 750-foot intervals opposite each other and upstream and downstream of structures as directed. Also, safety cables are installed upstream of all in-line canal structures.

SUMMARY

This paper presents the current guidelines and practice of the Bureau of Reclamation in its design and construction of concrete canal linings. The Bureau has found through experience and laboratory testing that concrete linings are effective in controlling seepage, weed growth, reducing flow resistance and maintenance costs.

REFERENCES

1. "Canals and Related Structures - Design Standards No. 3," Bureau of Reclamation, Release No. DS-3-5, December 1967

2. "Linings for Irrigation Canals," Bureau of Reclamation, First Edition, 1963

3. "Earth Manual," A Water Resources Technical Publications, Second Edition, 1974

4. "Measuring Seepage from Irrigation Channels," U.S. Department of Agriculture, Technical Bulletin No. 1203, September 1959

5. "Concrete Manual," Bureau of Reclamation, Seventh Edition, 1963

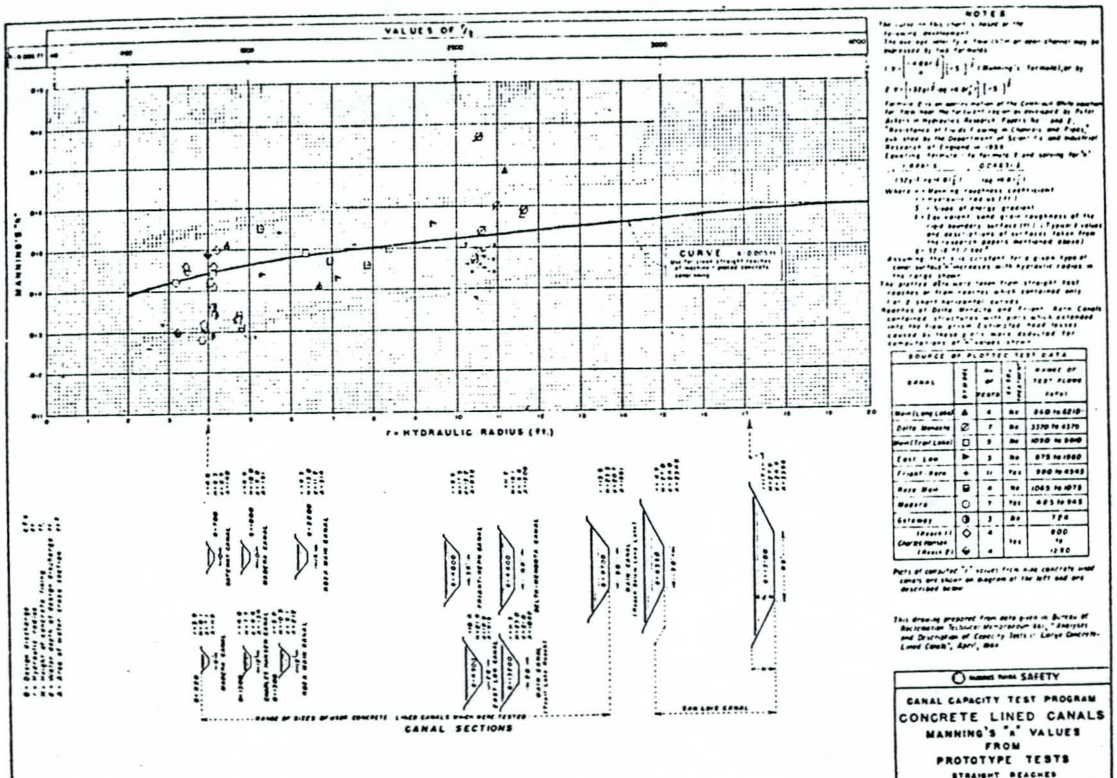


FIGURE 1

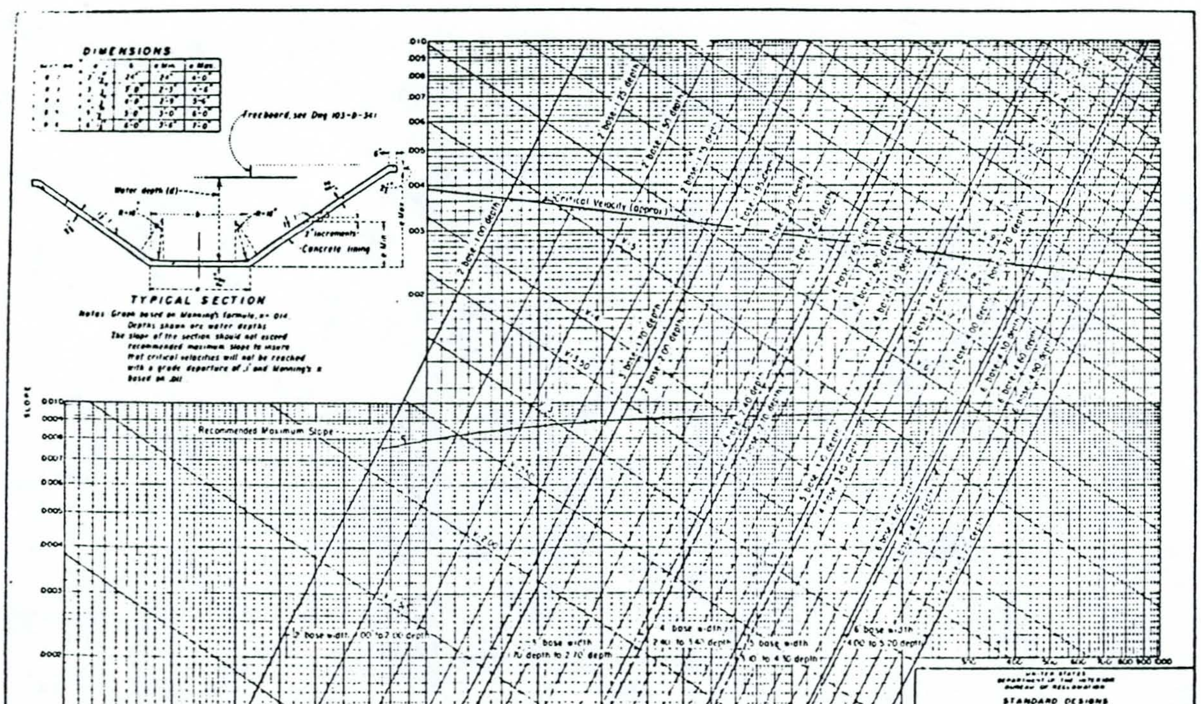


FIGURE 2

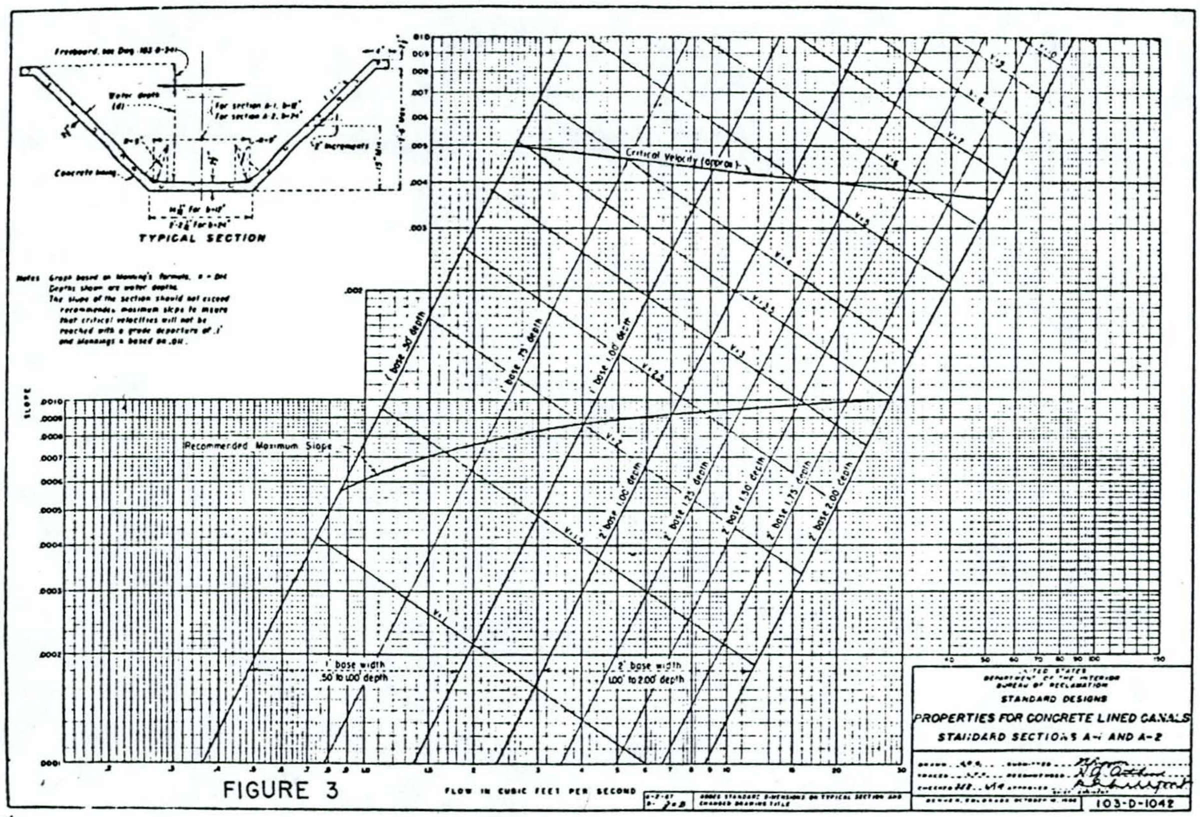


FIGURE 3

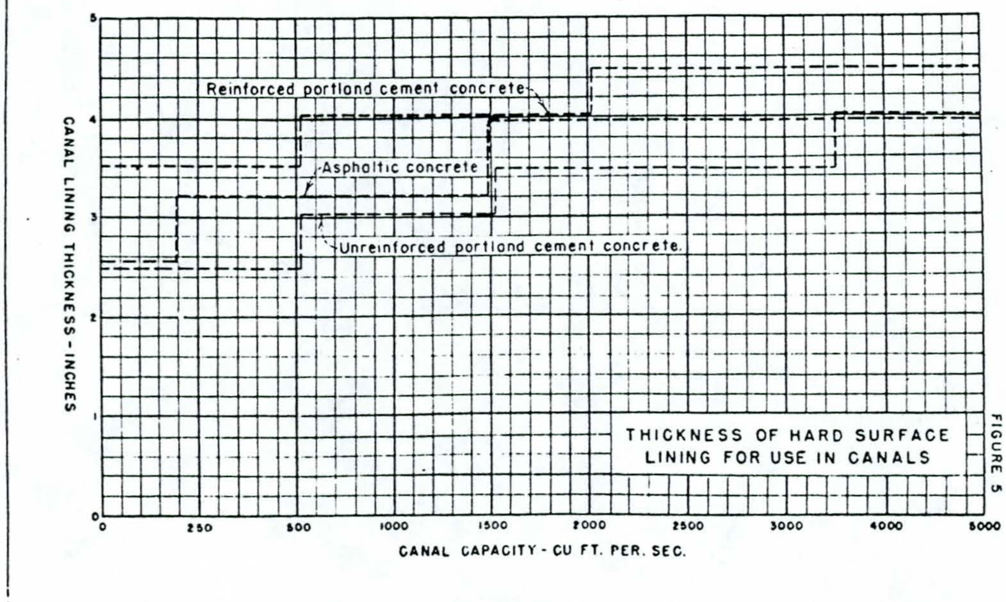
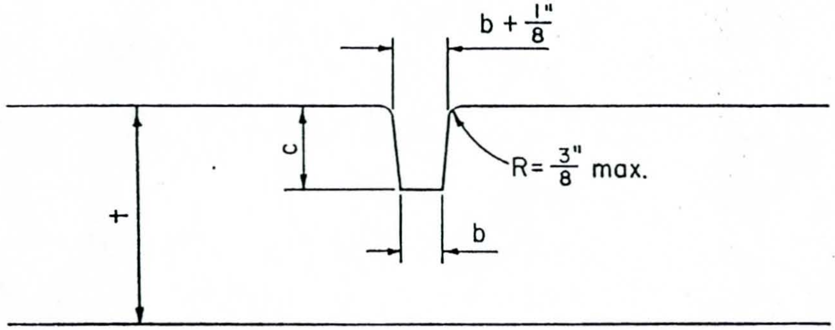


FIGURE 4



t, inches	b, inches	c, inches	Approximate groove spacing, center to center (feet – inches)
2	$\frac{1}{4}$ to $\frac{3}{8}$	$\frac{5}{8}$ to $\frac{3}{4}$	10-0
$2\frac{1}{2}$	$\frac{1}{4}$ to $\frac{3}{8}$	$\frac{3}{4}$ to $\frac{7}{8}$	10-0
3	$\frac{3}{8}$ to $\frac{1}{2}$	1 to $1\frac{1}{8}$	12-0 to 15-0
$3\frac{1}{2}$	$\frac{3}{8}$ to $\frac{1}{2}$	$1\frac{1}{8}$ to $1\frac{1}{4}$	12-0 to 15-0
4	$\frac{3}{8}$ to $\frac{1}{2}$	$1\frac{1}{4}$ to $1\frac{3}{8}$	12-0 to 15-0

Dimension b and c show allowable tolerance

FIGURE 5

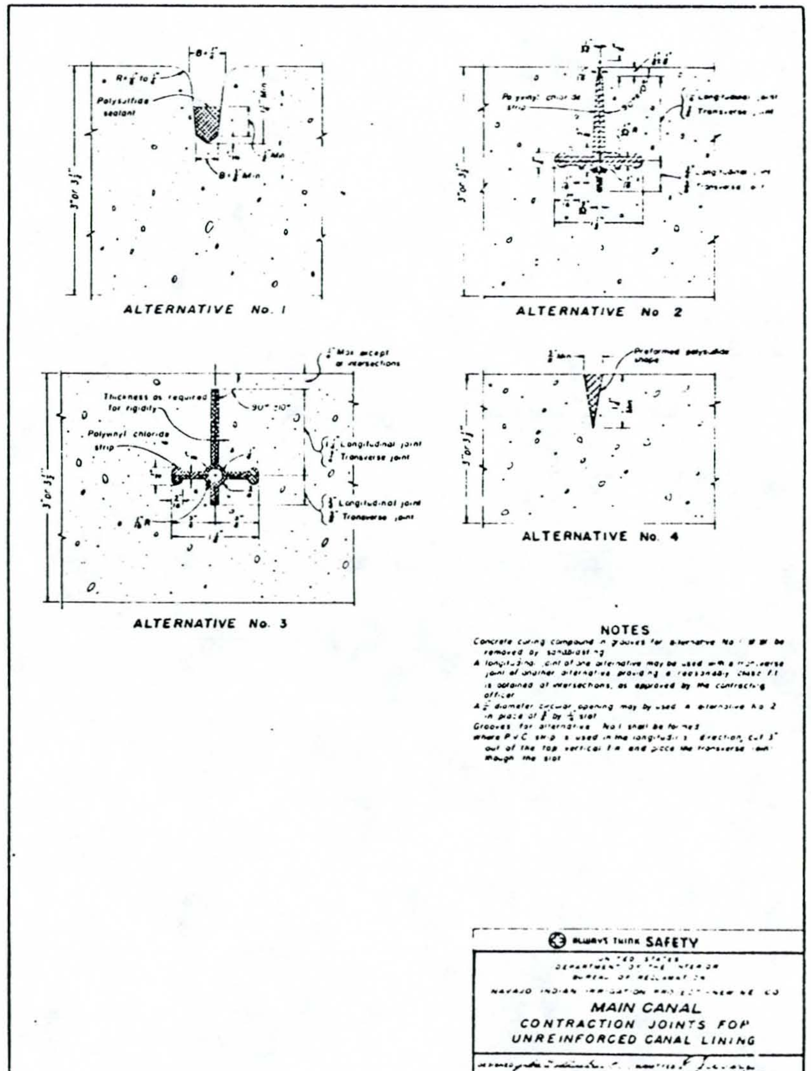
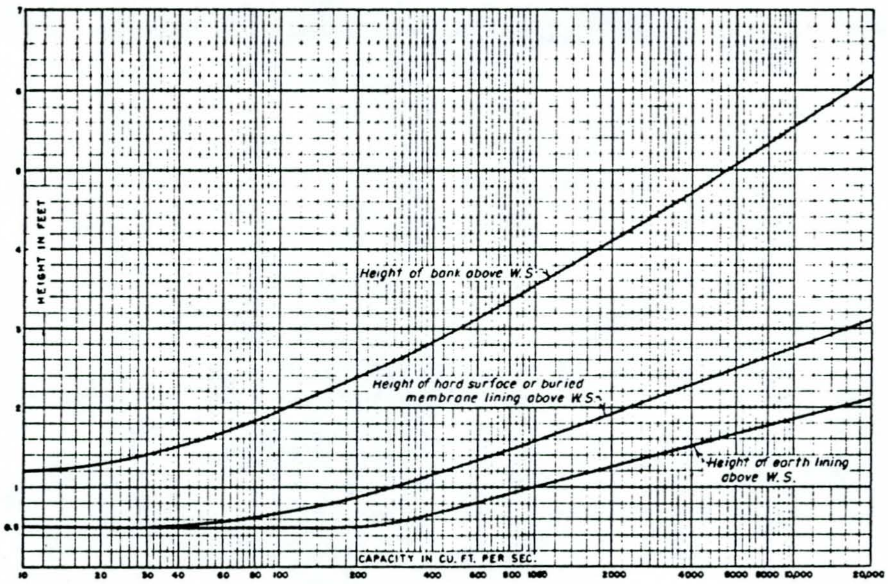


FIGURE 6



BANK HEIGHT FOR CANALS
AND
FREEBOARD FOR HARD SURFACE, BURIED MEMBRANE, AND EARTH LININGS

FIGURE 7

REVISED 3-10-60 103-D-341

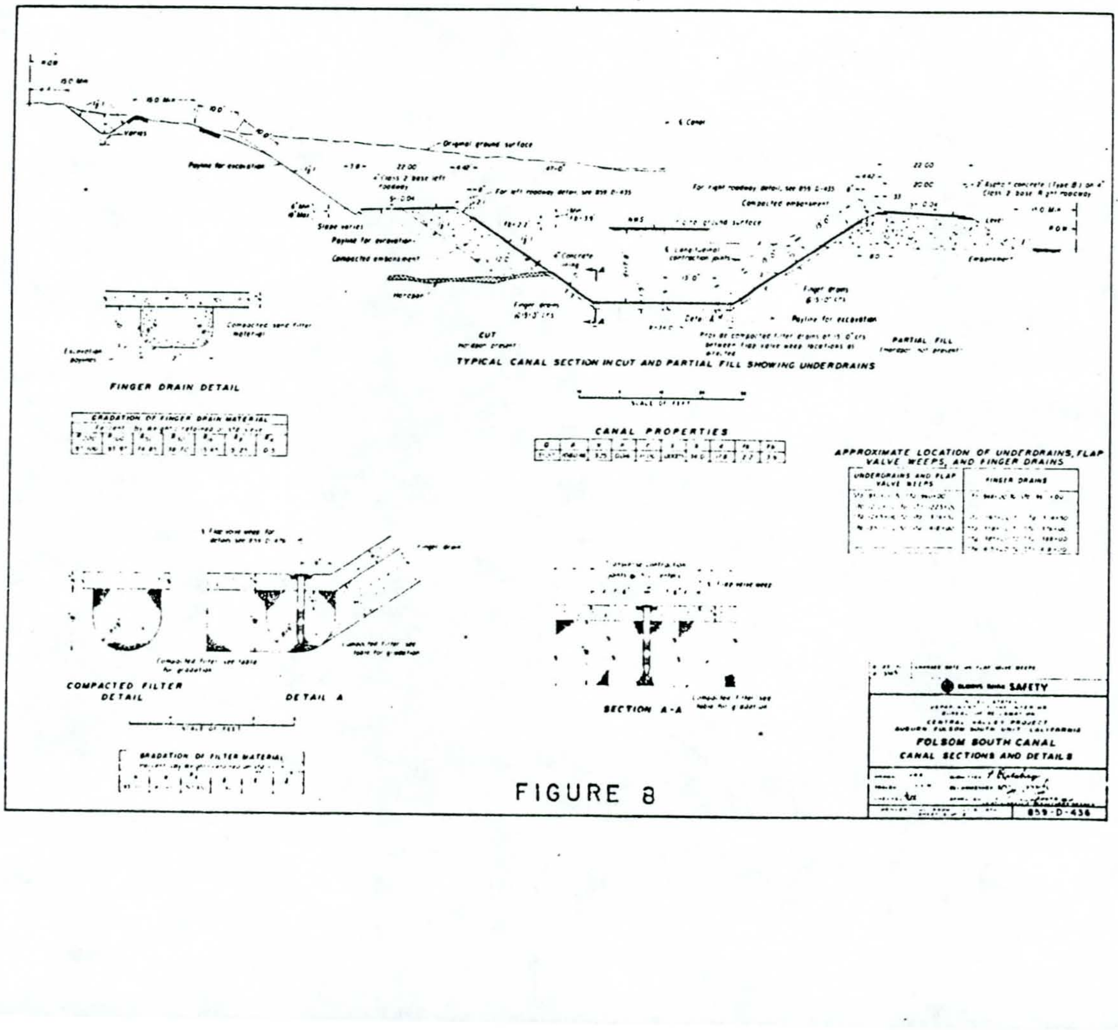


FIGURE 8

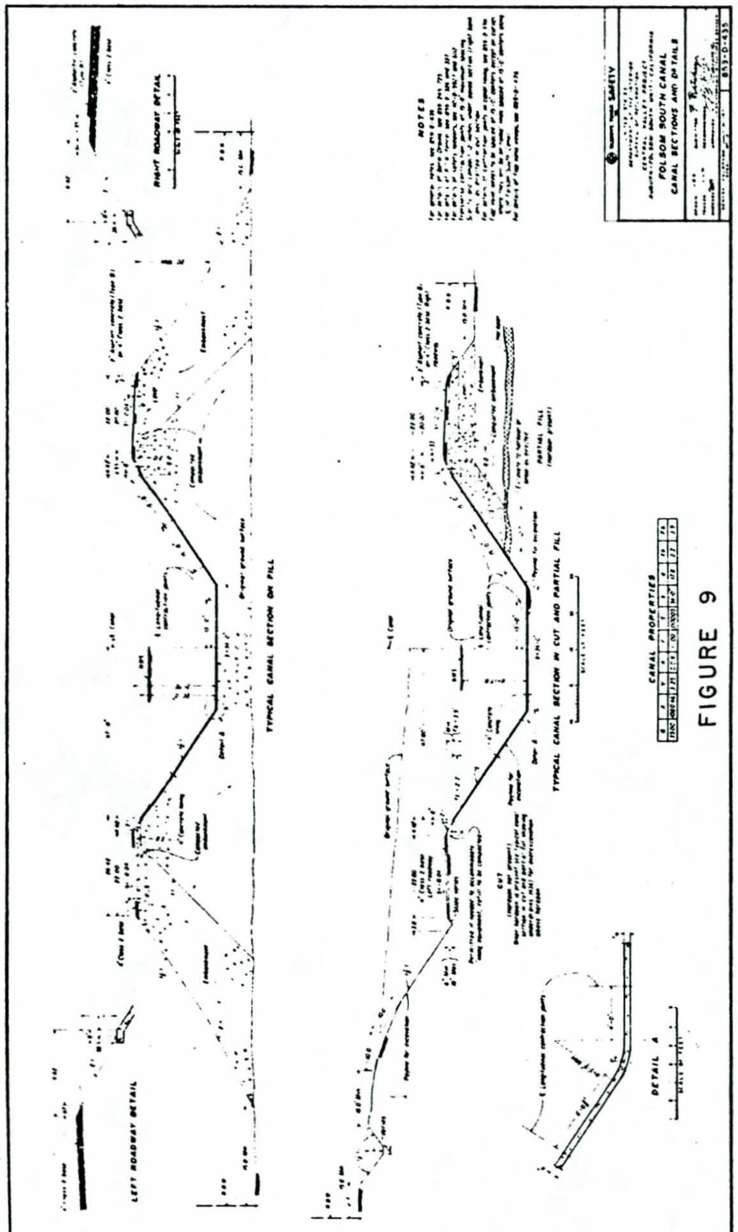
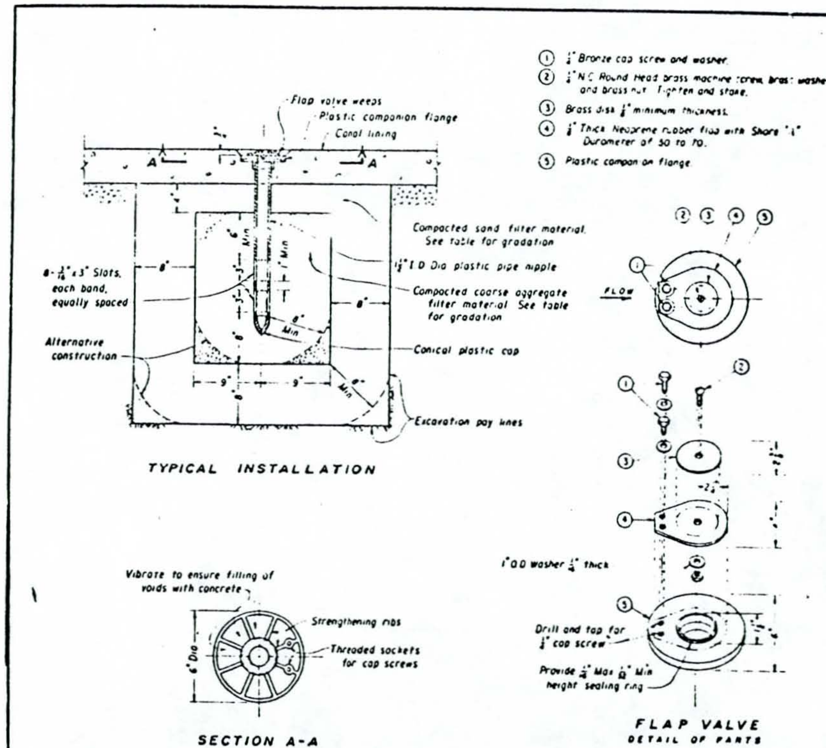


FIGURE 9



GRADATION OF FILTER MATERIALS

FILTER MATERIAL	PERCENT (BY WEIGHT) RETAINED ON STANDARD SIEVE									
	No 200	No 100	No 50	No 30	No 16	No 8	No 4	3/8"	3/4"	1½"
Sand	97-100	93-97	74-85	38-70	15-45	5-25	0-5			
Coarse Aggregate					95-100	75-90	60-80	40-60	20-35	0

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

FLAP VALVE WEEPS

DRAWN ... SUBMITTED ...
TRACED ... RECOMMENDED ...
CHECKED ... APPROVED ...
DENVER, COLORADO 103-D-IC-4

FIGURE 10

BUREAU OF RECLAMATION PRACTICES FOR DESIGN AND CONSTRUCTION OF CONCRETE LINED CANALS

By Forrest Ritchey, F. ASCE 1/
John G. Starbuck, M. ASCE 2/

Water losses in canals through seepage are reduced through the installation of portland cement concrete lining. Except in specific instances where canal safety is imperative and seepage must be reduced to near zero, the lining is not reinforced because of the high cost. Also, concrete linings eliminate weed growth, reduce flow resistance and maintenance costs, and are impervious to burrowing animals.

To determine the need for canal lining, consideration must be given to seepage rates, value of water saved, operation and maintenance costs, drainage costs or value of land taken out of cultivation by seepage if lining is not provided, canal size, reservoir size, right-of-way, allowable velocities, structure costs, and environmental factors.

Velocity:

The Manning formula is used by the Bureau of Reclamation for computations for open channel flow. The formula is as follows:

$$V = \frac{1.486}{n} r^{2/3} s^{1/2}$$

V = velocity of water in feet per second

S = slope of energy gradient in feet per foot

r = hydraulic radius (water area divided by wetted perimeter)

n = coefficient of roughness

The coefficient of roughness varies with the size of the canal and is shown on figure 1.

Usually velocities should be less than 8 feet per second to avoid the possibility of converting velocity head through a crack to pressure head under the lining and lifting the lining. A design check using an "n" value of 0.003 less than the design "n" used for the lining is also required to make certain the depth of flow does not approach critical

1/ Supervisory Civil Engineer, Head, Canal Structures and Bridge Section, Bureau of Reclamation, Denver, Colorado
2/ Civil Engineer, Bureau of Reclamation, Denver, Colorado

depth closely enough to develop standing waves at sections where the bottom might be raised above theoretical grade due to construction tolerances. The alinement for a concrete-lined canal should have a minimum radius of three times the water surface width.

Hydraulic Properties:

From experience, the steepest satisfactory side slope for a large lined canal from both construction and maintenance considerations is 1-1/2:1. Concrete-lined canals have a ratio of base width to water (B/D) depth of from 1 to 2. Small canals normally have a ratio of nearly 1, while large canals may have a ratio up to 2. The section with the least wetted perimeter is the most economical. Figures 2 and 3 show Bureau of Reclamation standard dimensions and hydraulic properties for small canals with concrete lining. Figure 4 shows normal lining thickness.

Foundation:

Concrete lining requires a firm, dense foundation of existing or compacted earth with a smooth surface to receive the lining. In low density areas the canal is overexcavated and backfilled with suitable compacted earth. When partial backfilling of an existing canal is necessary to reduce the cross-sectional area to that required for a lined canal, the backfill must be compacted to such a degree that subsequent settlement will not rupture the lining. In rock excavation except shale, the portion of the section to be covered with concrete lining is overexcavated so that there will be not less than 3 inches between rock points and the underside of the lining. Surfaces so excavated are filled with compacted pervious material. Where expansive clay or shale is encountered the canal prism is overexcavated a minimum of 2 feet below the underside of the lining, measured normal to the bottom and side slopes. The excess excavation is refilled to the underside of the concrete lining with selected compacted impervious material. Where lens or lenses of shale occur in the foundation the entire lining foundation is overexcavated above the bottom of the lowest shale lens 2 feet below the underside of the canal lining. The excess excavation is refilled to the underside of the concrete lining with selected compacted impervious material. The required degree of compaction varies with the soil, in-place materials, thickness of the backfill, and type of lining to be used. These related factors must be given special consideration for each such installation.

Concrete Lining:

Concrete canal lining is subject to complex stresses resulting from temperatures or moisture change in the slab or from a combination of the two.

The compressive stress resulting from either a temperature or moisture increase is of little concern for two reasons. First, a slab which is fully restrained at both ends and subjected to a 100° F increase in

U.S. ARMY CORPS OF ENGINEERS

√Struc. Sect.

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DAEN-CWE-D

DEPARTMENT OF THE ARMY
Office of the Chief of Engineers
Washington, D.C. 20314

ETL 1110-2-236
current on my ETL
~~25 Feb 83~~
22 Feb 84

Engineer Technical
Letter No. 1110-2-236

30 June 1978

Engineering and Design
DESIGN CRITERIA - PAVED CONCRETE FLOOD CONTROL CHANNELS

1. Purpose. This ETL provides guidance for the design of joints, reinforcement and subdrainage for concrete lined flood control channels.

2. Applicability. This letter applies to all field operating agencies having responsibility for the design and construction of Civil Works.

3. Reference.

a. EM 1110-2-2103

b. EM 1110-2-2400

c. EM 1110-2-2000

4. Discussion. Most of the Corps of Engineers experience with paved concrete channels has been in the Los Angeles District. However, in recent years Districts of the Southwestern Division have had some fairly extensive paved concrete channel projects. Designs for other recent projects prepared by various Districts indicate that guidance is needed to assure that all facets of the design are given proper consideration.

5. Subdrainage System.

a. The attached typical detail drawings for subdrainage systems should be used as guides where the permanent water table is above the channel invert or where it is reasonable to anticipate that temporary water tables due to seasonal variation or local ponding may be above the channel invert. The length of the closed drain needs to be designed so that excess hydrostatic head is not transferred to a downstream location. In this regard, the construction of the upstream soil plug is important. Selection of locations for soil plugs and drain outlets needs to consider the profile for maximum ground water surface. For cases where the water table will not be above the channel invert, a subdrainage system will not be required. In some cases (i.e., perched water conditions) a reduced subdrainage system commensurate with hydraulic and foundation conditions will be adequate.

b. The details for manholes and valves shown in the drawings are acceptable but more economical manhole designs are often adequate. Precast concrete pipe will usually be acceptable for use in manholes. Where an unusual amount of silt or debris is expected, access for clean-out of the drainage system is necessary. The details shown on the drawings will serve as an adequate clean-out design. In general, filter material under the side slope paving should not extend to more than half the height of the side slope paving and where foundation materials are suitable, the continuous filter under the side slopes may be replaced by finger drains as shown on the drawings.

c. A closed drainage system may not be required for short sections of channel paving or where the channel is paved only under bridges. For these cases an open drain system discharging through the slope paving is usually sufficient.

d. Where the paved channel is in rock, the slab may be anchored to rock and the drainage should consist of open holes drilled to the same depth as the anchors. The drainage system should provide for adequate control of aquifers at or near the top of bedrock and behind the side slope paving slabs.

6. Structural Design.

a. Concrete. All concrete should have maximum water-cement ratios as outlined in Table III (Reference 3c) or have a compressive strength of at least 3,000 psi at 28 days. For U-channels, the minimum thickness of the wall and invert slab should be not less than 10 inches and preferably 12 inches. Other rectangular shaped channels should also have a minimum thickness of wall stem and footing of 12 inches. The invert slab for both rectangular and trapezoidal shaped channels should generally be 10 inches and in no case, should it be less than 8 inches for main channel paving. The slope paving should generally be 8 inches and in no case, less than 6 inches for main channel paving. For small side channels, the invert and slope slab thickness should not be less than 6 inches and 4 inches, respectively. In cases where the paving stops below the level of the standard project flood, the edges of the paving at the top of the slope should be provided with at least a 2'-0" cutoff for the main channels. This cutoff may be 12 inches when the slope paving extends above the level of the standard project flood.

b. Joints. Past experience has shown that the usual sealing compound will not give satisfactory performance when subjected to any sand scouring. The Los Angeles District has had varied experience in the spacing and types of joint; in general, the experience of that District, which includes the construction of a large percentage of the channel linings constructed in the Corps over the past 25 years, has revealed that the more joints you have, the more maintenance is required. Based upon that experience, the paving for the invert and slopes of channels should generally be designed as continuously reinforced, without transverse and longitudinal joints, except construction joints placed at the end of a day's run; between adjacent paving lanes; or anytime the concreting operation has ceased for 45 minutes. For the rectangular shaped channels, contraction joints in the wall should generally be spaced at intervals of 40 to 50 feet. Vertical expansion joints should be provided only where the channel paving abuts another structure such as a box culvert, or bridge pier. Normally, the expansion joints should be provided with a waterstop to avoid overloading the subdrainage system.

c. Steel Reinforcement. Local temperature extremes is only one of several elements which determine the minimum amount of shrinkage and temperature reinforcement required. Composition of the concrete, drying shrinkage, moisture movement and subgrade movement contribute to cracks in concrete as well as local temperature extremes. However, it is essential to provide a slab of maximum flexibility which will accommodate settlement with the occurrence of only minute or harmless cracks. For slabs less than 12 inches in thickness, the steel should be located at the center of the slab. Within a slab, the cross-sectional area of the reinforcing steel (Grade 60 steel) should be about 0.5 percent of the cross-sectional area of the slab. The longitudinal steel for a continuously reinforced concrete slab should be about 0.30 percent of the gross cross-sectional area in regions with moderate climatic conditions and about 0.40 percent in regions with severe climatic temperature conditions. Transverse steel should be about 0.20 percent in regions with moderate climatic conditions and about 0.27 percent in regions with severe climatic conditions.

FOR THE CHIEF OF ENGINEERS:

Homer B. Willis

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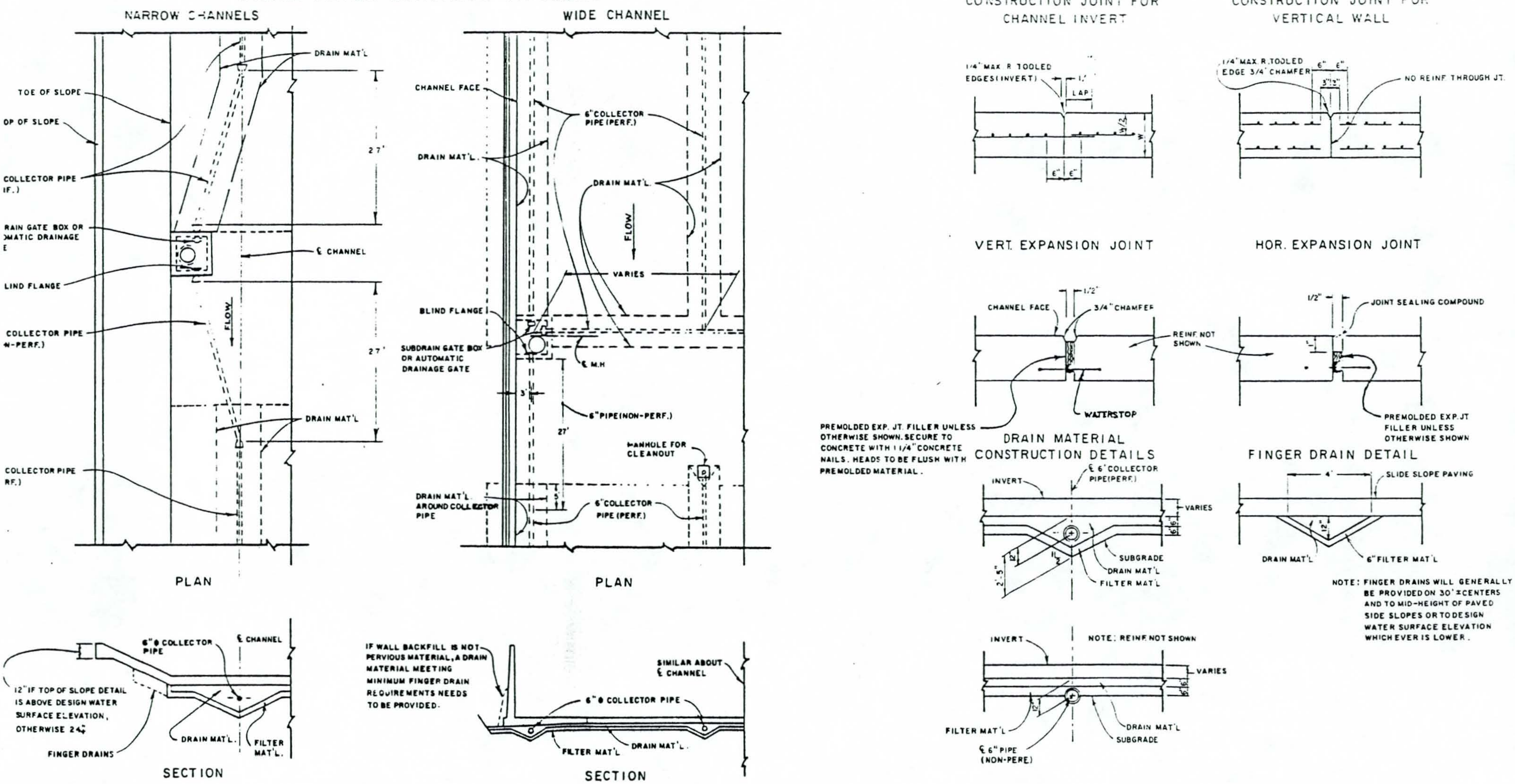
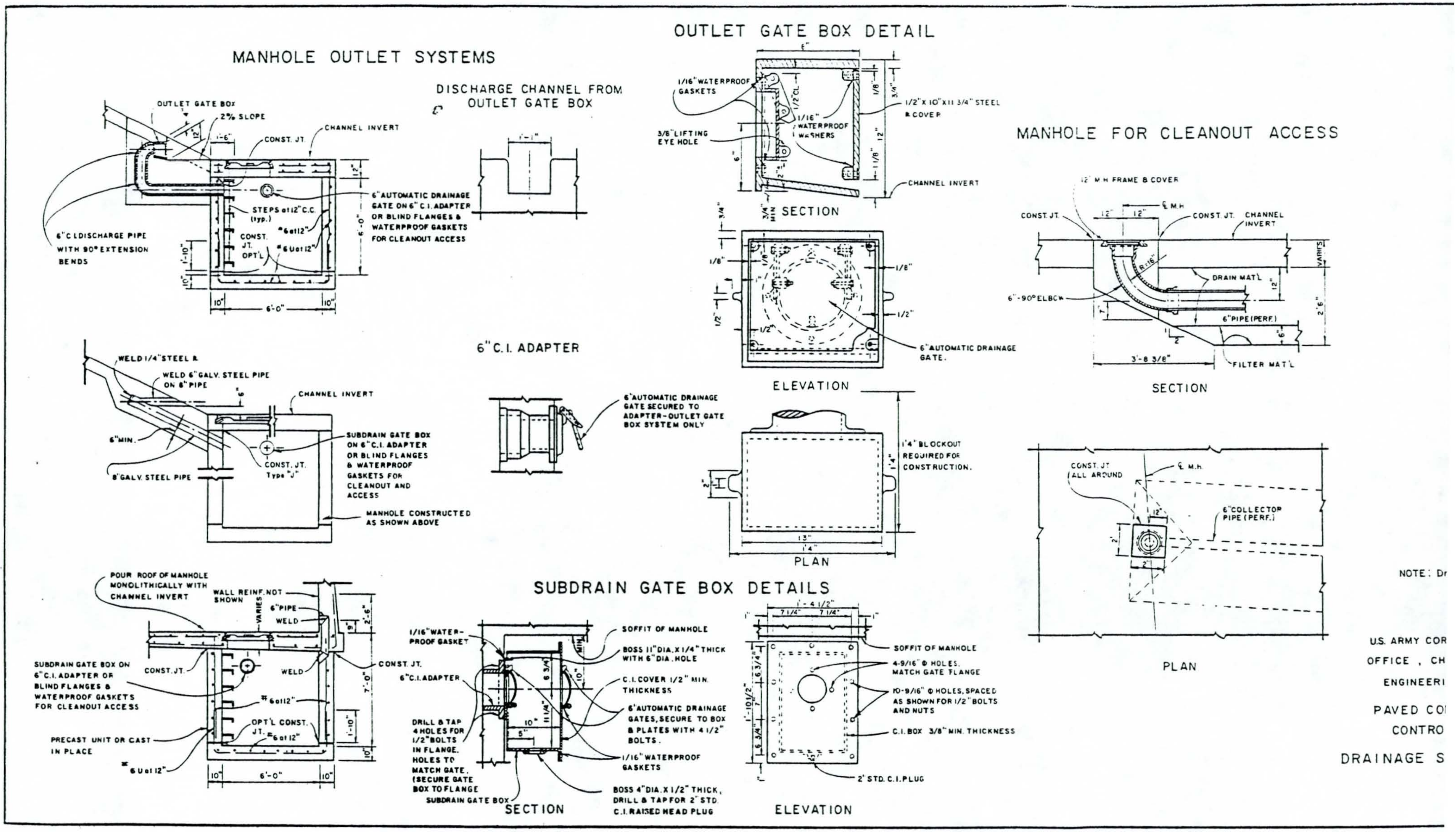
HOMER B. WILLIS
Chief, Engineering Division
Directorate of Civil Works

TYPICAL CONFIGURATIONS FOR DRAINAGE SYSTEMS UNDER PAVED CONCRETE CHANNELS




MANHOLE OUTLET SYSTEMS
OUTLET GATE BOX
2% SLOPE
CONST. JT.
CHANNEL INVERT
6" C.I. DISCHARGE PIPE WITH 90° EXTENSION BENDS
STEPS at 12" C.C. (typ.)
CONST. JT. OPT'L
6" AUTOMATIC DRAINAGE GATE ON 6" C.I. ADAPTER OR BLIND FLANGES & WATERPROOF GASKETS FOR CLEANOUT ACCESS
6'-0"
DISCHARGE CHANNEL FROM OUTLET GATE BOX
6" C.I. ADAPTER
WELD 1/4" STEEL PL
WELD 6" GALV. STEEL PIPE ON 8" PIPE
CHANNEL INVERT
6" MIN.
8" GALV. STEEL PIPE
CONST. JT. Type "J"
SUBDRAIN GATE BOX ON 6" C.I. ADAPTER OR BLIND FLANGES & WATERPROOF GASKETS FOR CLEANOUT AND ACCESS
MANHOLE CONSTRUCTED AS SHOWN ABOVE
6" AUTOMATIC DRAINAGE GATE SECURED TO ADAPTER-OUTLET GATE BOX SYSTEM ONLY
POUR ROOF OF MANHOLE MONOLITHICALLY WITH CHANNEL INVERT
WALL REINF. NOT SHOWN
6" PIPE
WELD
CONST. JT.
SUBDRAIN GATE BOX ON 6" C.I. ADAPTER OR BLIND FLANGES & WATERPROOF GASKETS FOR CLEANOUT ACCESS
OPT'L CONST. JT.
PRECAST UNIT OR CAST IN PLACE
OUTLET GATE BOX DETAIL
1/16" WATERPROOF GASKETS
3/8" LIFTING EYE HOLE
1/16" WATERPROOF WASHERS
1/2" X 10" X 11 3/4" STEEL PL COVER
CHANNEL INVERT
SECTION
6" AUTOMATIC DRAINAGE GATE.
ELEVATION
1'4" BLOCKOUT REQUIRED FOR CONSTRUCTION.
PLAN
SUBDRAIN GATE BOX DETAILS
1/16" WATERPROOF GASKET
6" C.I. ADAPTER
DRILL & TAP 4 HOLES FOR 1/2" BOLTS IN FLANGE. HOLES TO MATCH GATE. (SECURE GATE BOX TO FLANGE
SUBDRAIN GATE BOX
SOFFIT OF MANHOLE
BOSS 11" DIA. X 1/4" THICK WITH 6" DIA. HOLE
C.I. COVER 1/2" MIN. THICKNESS
6" AUTOMATIC DRAINAGE GATES, SECURE TO BOX & PLATES WITH 4 1/2" BOLTS.
1/16" WATERPROOF GASKETS
BOSS 4" DIA. X 1/2" THICK, DRILL & TAP FOR 2" STD. C.I. RAISED HEAD PLUG
SECTION
SOFFIT OF MANHOLE
4-9/16" Ø HOLES, MATCH GATE FLANGE
10-9/16" Ø HOLES, SPACED AS SHOWN FOR 1/2" BOLTS AND NUTS
C.I. BOX 3/8" MIN. THICKNESS
2" STD. C.I. PLUG
ELEVATION
MANHOLE FOR CLEANOUT ACCESS
12" M.H. FRAME & COVER
CONST. JT.
℄ M.H.
CONST. JT.
CHANNEL INVERT
DRAIN MAT'L
6"-90° ELBOW
6" PIPE (PERF.)
FILTER MAT'L
3'-8 3/8"
SECTION
CONST. JT. (ALL AROUND
℄ M.H.
6" COLLECTOR PIPE (PERF.)
PLAN
NOTE: Dr
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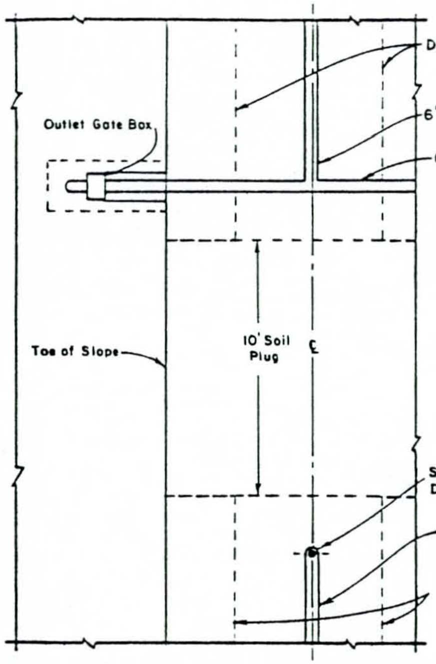
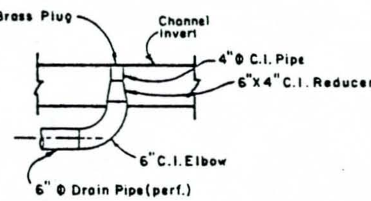
CONFIGURATION FOR A REDUCED DRAINAGE SYSTEM



PLAN

CLEANOUT DETAIL



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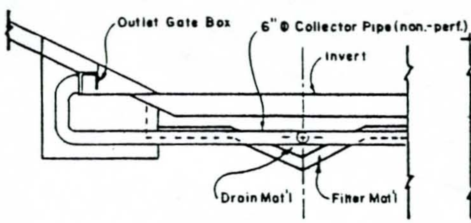
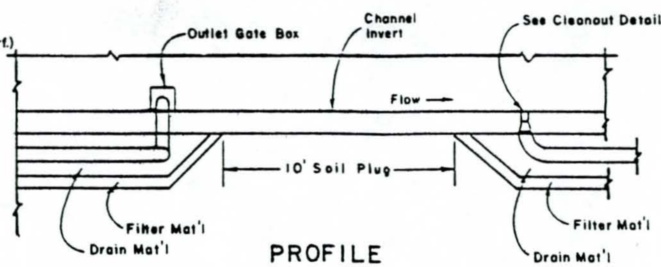


SECTION



PROFILE

US ARMY CORPS OF ENGINEERS
OFFICE, CHIEF OF ENGINEERS
ENGINEERING AND DESIGN

PAVED CONCRETE FLOOD CONTROL CHANNELS

REDUCED DRAINAGE SYSTEM
CONFIGURATION AND DETAILS

Drawing No. 3

ENGINEER MANUAL

EM 1110-2-1603

31 MARCH 1965

ENGINEERING AND DESIGN

HYDRAULIC DESIGN OF SPILLWAYS

HEADQUARTERS, DEPARTMENT OF THE ARMY

OFFICE OF THE CHIEF OF ENGINEERS

to determine the relation.

In view of the Ft. Randall tests, design n values of 0.015 and 0.013 are recommended for determination of sidewall heights and stilling basin entering velocities, respectively. It should be emphasized that other factors, as described in paragraph 23, must also be taken into account in establishing sidewall heights. More prototype test results are needed to afford a more reliable basis for other absolute roughness values and Froude numbers.

d. Chute Floor. Important details of the spillway chute floor slabs deserve careful attention in the interests of structural safety and economy. The structural aspects are discussed in EM 1110-2-2400 and in reference 75. There are three important hydraulic phenomena which can produce uplift under the floor slab, and at least one which can produce severe negative pressures on the top of the slab. Two are static-type phenomena and two are hydrodynamic.

(1) Uplift from reservoir pressure. In the case of gated control structures where the reservoir operating pool can stand high on the crest gates, a substantial uplift can exist on the slab just downstream from the toe of the control structure. The intensity of the uplift pressure involves the same considerations as uplift pressures under concrete dams. Both the principles of estimating the pressure intensity and the protective measures involved are treated extensively in the technical literature.

(2) Uplift from the hydraulic jump. Uplift on slabs in a stilling basin and on the slope approaching the stilling basin can occur in a manner similar to the hydrostatic case discussed immediately above. In this case, the difference in elevation between the sequent depths d_1 and d_2 can transmit a pressure from the downstream tailwater through joints or cracks to the underneath side of the slab upstream. The hydrodynamic principles involved are presented in paragraph 12. The phenomena are more severe than the uplift occasioned by a gated control structure because of pulsating pressures in the toe of the jump or from the rapid unstable flow entering the jump. Such pulsations could conceivably induce small

relative movement across transverse joints.

(3) Control crest, chute junction. Actually two separate phenomena can combine to produce high uplift through a joint at the junction between a low ogee crest and the spillway chute slab. The boundary pressure caused by the circular toe curve of the ogee crest is discussed in subparagraph 21b. It should be noted that the pressure does not reduce to that attributable only to the depth of flow at the point of tangency with the chute floor. If the toe curve radius is too small and the total drop from the reservoir elevation is considerable, a substantial boundary pressure can exist at the point of tangency.

In case of relative movement at the joint, so that the edge of the slab is higher than the boundary upstream from the joint, a substantial pressure can be transmitted into a joint opening. Such displacement could occur from a small rotation of the crest under the hydrostatic load or from foundation movement under the slab. The uplift so induced is indicated schematically in Plate 39(a). At the location under consideration, the turbulent boundary layer may not have developed to any great thickness so that a conservative boundary velocity for design purposes would be the mean velocity based on potential flow. This assumes no head loss from the reservoir level and the mean velocity can be estimated by the use of specific energy diagrams such as those in HDC 123-2 through -5. The toe curve pressure and the boundary velocity induced pressure are additive.

An example of the damage which can be caused by this type of uplift is the failure of the chute slabs at Dickinson Dam.[95] Four 15-in.-thick slabs adjacent to the ogee crest toe were dislodged and part of the slab foundation eroded. The maximum head over the crest was only about 3 ft when this occurred. The total energy head from reservoir water surface to the end of the toe curve was about 7.5 ft. There was probably also a residual boundary pressure caused by the toe curve. Subsequent inspection of the failure indicated that the slab subdrains were frozen to an extent that prevented their proper functioning. The hydraulic problem of the efficacy of gravel blankets with perforated pipe in reducing high pressures needs further investigation. The same uplift phenomena probably account for the displacement of large slabs of rock as described in subparagraph 15e. An offset should be used at slab joints in high-velocity chutes, as shown in Plate 39(b), to prevent uplift if joint displacements are likely to occur.

(4) Vertical curves. A break in grade of the profile from a light slope to a steeper slope will cause a negative pressure on the slab unless the vertical curve is properly designed. The design is similar to that for a parabolic drop from a tunnel exit portal to a stilling basin floor. The equation for the trajectory of a jet is given in EM 1110-2-1602, subparagraph 25c. The velocity used for computing the curve should be 25 percent greater than the estimated mean velocity of design flow.

23. SIDEWALLS. The height of chute spillway sidewalls should be designed to contain the flow for the spillway design flood. In addition to the determination of the water-surface profile, allowance should be made for pier end waves, slug flow or roll waves, and air entrainment.

a. Water-Surface Profile. In most cases the flow will not enter the spillway chute at normal depth. It is therefore necessary to compute the water-surface profile by methods dealing with varied flow. One step method which has been used is given in Appendix IV of EM 1110-2-1602. The use of varied flow functions greatly simplifies the computation. The Bakhmeteff varied flow equation[96] is often used for chute spillway design.

$$L \frac{y_o}{S_o} = (\eta_2 - \eta_1) - (1 - \beta) [B (\eta_2) - B (\eta_1)] \qquad (22)$$

where L is the distance between sections 1 and 2, y_o the normal depth, and S_o the slope of the floor. The two dimensionless coefficients are

$$\eta = \frac{y}{y_o}$$

U.S. SOIL CONSERVATION SERVICE

TECHNICAL RELEASE

NUMBER 67

REINFORCED CONCRETE STRENGTH DESIGN

AUGUST 1980

U. S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

NATIONAL ENGINEERING STAFF

DESIGN UNIT

Details of Reinforcement

crete Cover for Reinforcement

e minimum clear concrete cover over reinforcement is two inches except that he minimum clear cover is three inches when the concrete is deposited on or against earth. In the structural design of slabs or beams without web reinforcement, the distance from the surface of the concrete to the centerline of the nearest reinforcing steel may be taken as 2 1/2 or 3 1/2 inches, as the case may be, to simplify the determination of the effective depth, for all bars one inch or less in diameter.

Consideration should be given to increasing the concrete cover when the surface of a slab will be exposed to high flow velocities and the water carries abrasive materials.

Temperature and Shrinkage Reinforcement

Reinforcing steel is required in both faces and in both (orthogonal) directions in all concrete slabs and walls, except that only one grid of reinforcing is required in concrete linings of trapezoidal channels. The steel serves either as principal reinforcement or as temperature and shrinkage reinforcement. The function of temperature and shrinkage reinforcement is not to eliminate cracks, it is to induce a sufficient number of small cracks so that no crack has excessive width. Well laid out temperature and shrinkage steel also serves the important auxiliary function of tying the structure together.

Where principal steel is required in only one direction, it shall ordinarily be placed nearer the concrete surface than the temperature steel. Where principal steel is required in both directions, the steel that carries the larger moment shall ordinarily be placed nearer the concrete surface. Where principal steel is required in neither direction, the temperature steel parallel to the longer dimension of the slab or wall will ordinarily be placed nearer the concrete surface.

The minimum steel area, for slabs and walls having thickness equal to or less than 32 inches, in each face and in each direction, expressed as the ratio, ρ_t, of reinforcement area, A_s, to gross concrete area, bt, are as follows:

Steel in the direction in which the distance between expansion or contraction joints does not exceed thirty feet,

ρ_t = 0.002 in the exposed face

ρ_t = 0.001 in the unexposed face.

Steel in the direction in which the distance between expansion or contraction joints exceeds thirty feet,

$\rho_t = 0.003$ in the exposed face

$\rho_t = 0.002$ in the unexposed face.

The minimum steel area for slabs and walls having thicknesses greater than 32 inches shall be computed as though the thickness were 32 inches

When expansion or contraction in a member is restrained along any line, the concept of equivalent distance between expansion or contraction joints should be used to determine the required steel ratio, ρ_t. The equivalent distance is taken as double the perpendicular distance from the line of restraint to the far edge or line of support of the member.

When the surface of a wall or slab will be exposed for a considerable period during construction, the steel provided should satisfy requirements for an exposed face.

Where a single grid of reinforcement is used, as permitted above, the steel ratio, ρ_t, shall be the sum of that listed for both faces.

Splices and development lengths for temperature and shrinkage reinforcement shall be designed for the full yield strength, f_y.

Spacing of Reinforcement

The maximum spacing of principal steel shall be twice the thickness of the slab or wall, but not more than 18 inches. The maximum spacing of temperature and shrinkage steel shall be three times the thickness of the slab or wall, but not more than 18 inches.

The clear distance between parallel bars in a layer shall not be less than the bar diameter, 1 1/3 times the maximum size of the coarse aggregate, nor 1 inch.

Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer. The clear distance between layers shall not be less than 1 inch.

The clear distance between bars also applies to the clear distance between a contact lap splice and adjacent splices or bars.

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Regional Technical Service Center
Engineering and Watershed Planning Unit
Portland, Oregon

This edition has been revised and enlarged to more effectively meet the needs of engineers in the thirteen states for design standards to supplement sections of the National Engineering Handbook.

These standards are a consolidation of design decisions and practices that have been developed by the engineering profession over a number of years. These have been and are accepted through application by Service engineers in the states served by the Portland Engineering and Watershed Planning Unit. In addition to the standards several design aids have been included to assist designers in the application of the standards.

It is intended that the manual will be expanded as technical resources permit to cover other types of engineering design commonly encountered in Service work. This manual will be reviewed and revised periodically as experience indicates changes to be desirable. Suggestions for its improvement are solicited.

This manual was compiled and edited by Harry Firman in collaboration with other members of the Engineering and Watershed Planning Unit design staff under the direction of Frank Muceus. These men received valuable informal assistance from many field engineers.

Edwin J. Moore
Head, E&WP Unit

October 1970

ENGINEERING DESIGN STANDARDS

ENGINEERING AND WATERSHED PLANNING UNIT
PORTLAND, OREGON

INTRODUCTION

Soil Conservation Service Engineering Handbook Sections constitute basic Service technical standards.

This manual, Engineering Design Standards, contains some restatement of frequently used material found in the Engineering Handbook. Most of the standards presented supplement or modify the Handbook to provide more complete and applicable criteria for design in this area. This manual, with the Handbook, is intended to establish the general quality of design for the Service. It is not intended to limit initiative and resourcefulness of the engineer in developing plans consistent with adequacy of design indicated by these standards.

Explanation and background for these standards will be found in referenced sections of the Engineering Handbook and other referenced sources.

Special consideration is to be given to local and state codes by adjustment of these minimum design criteria where such codes are more restrictive.

"~~A single~~ grid of reinforcement can be used as ~~set forth by NEH 6 for concrete linings of trapazoidal channels and~~ on minor structures. Minor structures are ~~defined as (1) having a concrete quantity of~~ less than 5 ~~cubic yards~~, and (2) having a site ~~location permitting~~ economical maintenance or replacement."

e. Section thickness

Minimum section thickness for structural members shall be 6 inches with a single grid of reinforcing steel. Minimum thickness for formed sections containing two grids of steel will be 8 inches. Formed sections thicker than 8 inches shall have a steel grid in each face with proper concrete cover. Tapered walls containing two grids of steel shall have a minimum top width of 8 inches.

Minimum section thickness based on hydraulic consideration shall be:

(1) Channel lining other than chute spillways . . . . . see Figure 1.7.

(2) Chute spillway

(a) Invert

Discharge/ft width - cfs	Lining thickness - inches
less than 100	9
100 to 250	12
greater than 250	15

(b) Sidewalls - satisfy moment requirement.

(3) Outlet structures (SAF basin - for use in planning and preliminary design) . . . . . see Figure 1.8. Reduction in thickness as given in this figure must be supported by analysis of actual site conditions.

f. Contraflexure

Structural members in which both positive and negative moments exist may have a single grid of reinforcement to satisfy the larger moment provided the smaller moment does not exceed the following:

Section thickness - inches	Moment ft-lbs/ft-width
6	540
7	735
8	960
9	Double grid required

g. Fillets, haunches

When a fillet extends a support to 10% of the span length, measured from center to center of supports, the moment at the middle of the span may be reduced due to haunch effect.

(1) Class 3000 concrete 0.45 f'c, Figure 1.1.

(2) Stirrup spacing, Figure 1.5.

5. **[illegible] Concrete**

Nonreinforced concrete channel lining with discharges less than 100 cfs and velocity less than 15 fps shall have the following minimum lining thickness:

a. Rectangular channels . . . . 3 1/2 inches

b. Trapezoidal channels

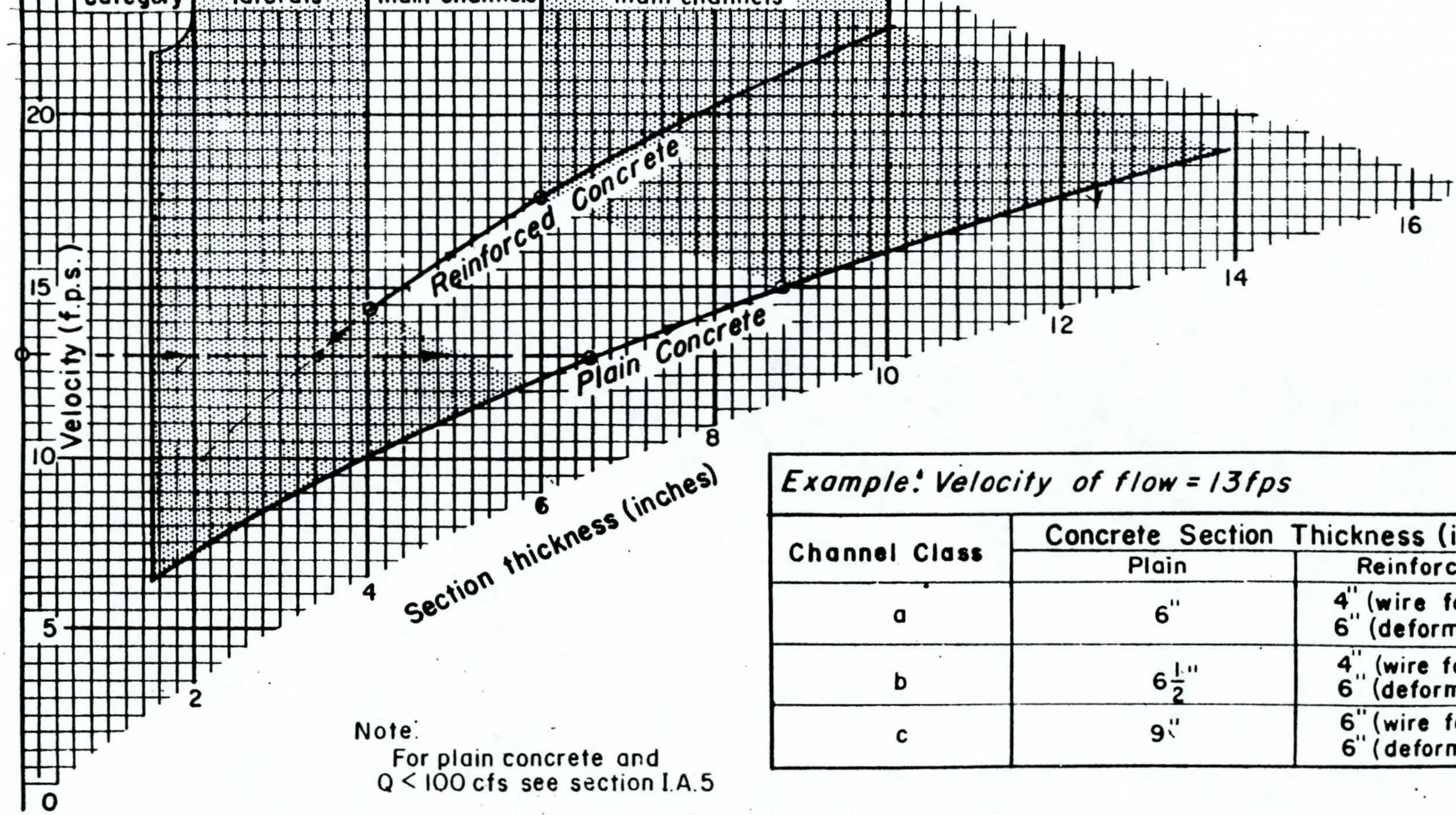
Design velocity	Climatic Area*		
	Mild	Moderate	Severe
ft. per sec.	Minimum Thickness Inches		
less than 6.0	1.5	2.0	2.5
6.0 to 9.0	2.0	2.0	2.5
9.0 to 12.0	2.5	2.5	2.5
12.0 to 15.0	3.0	3.0	3.0

* Climatic Areas

Mild - Average January temperature above 40°F.
Moderate - Average January temperature between 25°F and 40° F.
Severe - Average January temperature below 25°F.

For flow capacities in excess of 100 cfs . . . see Figure [illegible]

TITLE	LIMITATION		
Channel class	a	b	c
flow dura.	long (irrig.)	short	short (infrequent)
access.	divertible	uncontrolled	uncontrolled or below tide
category	laterals	main channels	main channels



Channel Class

a – minor development - rural area

b – moderate development - suburban area

c – intensive development - urban or industrial area

Subgrade shall be well drained

Note:
For plain concrete and $Q < 100$ cfs see section I.A.5

Example: Velocity of flow = 13fps

Channel Class	Concrete Section Thickness (inches)	
	Plain	Reinforced
a	6"	4" (wire fabric) 6" (deformed bars)
b	$6\frac{1}{2}$"	4" (wire fabric) 6" (deformed bars)
c	9"	6" (wire fabric) 6" (deformed bars)

FIGURE I.7
MINIMUM SECTION THICKNESS
Channel Lining

B. STRUCTURAL DESIGN - LINED CHANNELS

TR-50 is available for performing structural designs of rectangular channels. Many designs alternatives can be investigated rapidly.

1. Loads

   a. Base Slab. Soil pressures on base slabs for monolithic sections may be assumed as uniformly distributed for widths of channel less than 15 feet.

      For channels in excess of 15 feet, the elastic foundation approach may be used. See "Beams on Elastic Foundations", M. Hetenyi, University of Michigan Press, Ann Arbor, Michigan, 1946.

   b. Side Wall. Lateral pressures will be as determined from criteria listed in Section II.

2. Sub Drains

   All lined channels through earth where ground water is encountered should be adequately drained to relieve uplift pressures.

   Alternates are:

   1. Continuous drainage blanket weeped through the channel invert.

   2. Tile in gravel envelope outletting into the channel at infrequent intervals.

   All drains will be designed to serve as a proper filter.

   Rectangular channel linings having a bottom width less than 20' may be designed to resist full flotation forces by extending the invert in the form of wall heels.

3. Filter Design

   The Filter shall be designed as outlined in ~~See National Engineering Handbook, Section 11, page 6.5.~~ Soil Mech. Note No. 1.

4. Weep Holes

Minimum size weep holes shall be 2-1/2" diameter formed by non-corrosive liner.

5. Section Analysis

a. Rectangular Section. Economic section will be based on channel width-depth relation and lateral pressures.

Channels less than 20' wide shall be designed as "U" shaped structures.

Channels more than 20' wide may be designed as "U" sections, or as "L" or "T" sections with connecting invert slab acting as paving.

b. Trapezoidal Section. Trapezoidal sections, except in transitions, should be limited to paving.

6. Critical Design Conditions

Typical rectangular sections will be analyzed for conditions of:

a. Pre backfill - no lateral loads with weight of side walls on base slab using 150% of developed working strength of the concrete in the base slab at the time side walls are poured. See Figure 1.2.

b. Post backfill - channel empty. Lateral earth loads for soil type, backfill condition, and surcharge, as determined by tables, graphs or methods listed in Section II, Loads.

c. Post backfill - design flow.

(1) Tile collector or weep drained subgrade and backfill net equivalent fluid pressure on channel side from design water surface elevation. 40 pcf

(2) Without effective subdrain outletting into the channel, net equivalent fluid pressure on the channel side from the top of lining. 62.5 pcf

7. Temporary Construction Loads

Refer to Section 1, paragraph 4. i.

8. Minimum Section Thickness

a. Structural reinforced concrete sections.

Base slabs may be tapered for economy to reduce thickness in center section of invert as long as design requirements are met.

In channels carrying an abrasive bed load or velocities of flow in excess of 10 fps., a minimum of 3" cover shall be provided over the top steel in the invert. The additional 1" thickness shall not be used in computations as increasing effective depth.

b. Non structural, reinforced concrete section

Paving, wire mesh reinforcing	4 in.
deformed bar reinforcing	5-1/2 in.
Channel and chute lining, see page 1-3 and	Fig. 1.7
Stilling basins, see	Fig. 1.8

c. Asphalt concrete (temporary construction) 3" min.

d. Pneumatically applied mortar (See Fig. 1.7) 3" min.

9. Transverse Slope

Inverts in lined channels wider than 10' shallhave a transverse slope toward centerline. The drop in invert from the walls to centerline shall be 0.4 inch per linear foot.

10. Steel Placement

Steel should be detailed to facilitate placing, form removal, and to meet structural requirements. See Figure 1.3 for methods of providing continuity.

Reinforcing on curves shall be placed radially with design spacing c-c of bars on the outside of the curve.

11. Joints *

Reinforced concrete lining with expansion joints at maximum 30' spacing shall be reinforced longitudinally as discontinuous lining.

All lining with joint spacing in excess of 30' shall require longitudinal reinforcing as specified for continuous sections.

For linings designed as continuous sections expansion joints will be required at structures.

Paving slabs shall be doweled to each other and to toe of structural side walls with 5/8" smooth dowels 30" long spaced at 12" centers. Dowels shall have one end wrapped with tar paper or dipped in asphalt paint.

Reinforcing shall not be continuous through expansion joints.

Transverse joints in wall and slab shall be in the same vertical plane normal to channel centerline.

In discontinuous paving slabs weakened plain contraction joints shall be provided at not more than 30' spacing. Reinforcing shall not extend through the joint. Slabs shall be doweled as specified above.

* Refer also to the method in Appendix A for joint spacing.

12. Maintenance Roadways

Maintenance roadways should be provided on each side of channels in excess of 20' top width and on one side of smaller channels.

Maintenance roadways should have a minimum of 13' width with a 6" transverse slope away from the channel where the surrounding topography slopes toward the channel.

13. Berms and Interception Ditches

Interception ditches will be provided on both sides of the channel improvement throughout the length of the project to control local drainage, except through street rights-of-way. Bank slopes not topped with a roadway shall be sloped toward the right-of-way line to form an interception ditch. The top of the ditch bank thus formed will be at least 12" above natural ground.

Ditches shall be graded toward collecting points to drain into the channel through pipe overpour structures, or lined chute sections.

14. Cutoff Walls

In rectangular concrete lined channels cutoff walls are normally used at inlet and outlet terminals of the lining and at expansion joints.

In design of thin section trapezoidal linings cutoff walls should be used at the above locations and at intervals not to exceed 1000' to protect against progressive failure of the lining.

15. Soil Tests

Sufficient soil borings and lab tests shall be made to indicate the nature and extent of type of material to be encountered, suitability of the material for backfill, and the ground water elevation at the time of sampling. Seasonal variation in ground water as indicated by soil conditions and site locations should be appraised. Test holes should extend at least 2' below invert elevation with additional depth where questionable foundation conditions are encountered.

Test hole locations shall be shown on a plan and profile drawing with stratification and classification of materials logged. Dated water table (within depth of boring) shall also be logged.

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Engineering and Watershed Planning Unit
Portland, Oregon

ENGINEERING DESIGN STANDARDS

FAR WEST STATES

APPENDIX A

REINFORCEMENT OF CONCRETE FOR SHRINKAGE AND TEMPERATURE STRESSES IN CHANNEL LINING, RETAINING WALLS, AND PAVING SLABS

June 1966

REINFORCEMENT OF CONCRETE FOR SHRINKAGE AND TEMPERATURE STRESSES IN CHANNEL LINING, RETAINING WALLS, AND PAVING SLABS

Introduction

Reinforcement of concrete channel lining to satisfy structural demands is ordinarily not required except in the transverse direction of rectangular channels. Instead, a small amount of reinforcement is provided to maintain surface continuity and reduce the width of cracks.

Although the procedure presented in this Appendix is based on rational analysis of certain stresses, the reinforcement so provided is not limited in use to the stresses caused by concrete shrinkage or temperature changes but will restrict crack size and spacing within its capability regardless of the crack causing condition.

Types and Causes of Cracks in Concrete

Cracks in concrete are caused by conditions outlined in Figure A-1. The designer's control of these causes is limited to those parts of items No. 5, 6, 7, and 8 of Figure A-1 briefly described below.

Item No.	Cause	Description	Corrective Measure
5 C	Foreign bodies and rust	Oxidation of steel	Adequate cover and dense concrete
6 C	External temperature variation	Outside temperature	Expansion and contraction joints at adequate spacing
6 D	Frost and ice action	Penetration of water into	
		(a) concrete	(a) Provide dense concrete
		(b) subgrade	(b) Subgrade treatment
7 A	Reinforcement	Hairline cracks	Do not exceed elastic limit of concrete; provide for adequate bond

Item No.	Cause	Description	Corrective Measure
7 B	Structural form	Restriction of movement	Use keyed joints and locate cutoffs between joints; articulate
8	Structural design	(a) Design load	(a) Check assumption and analysis including uplift and swelling subgrade
		(b) Foundation settlement	(b) Design for tolerable amount or provide subgrade treatment

Emphasis on foundation preparation and treatment rather than increase in brute strength of the concrete could result in a better job and construction economy.

Design Criteria

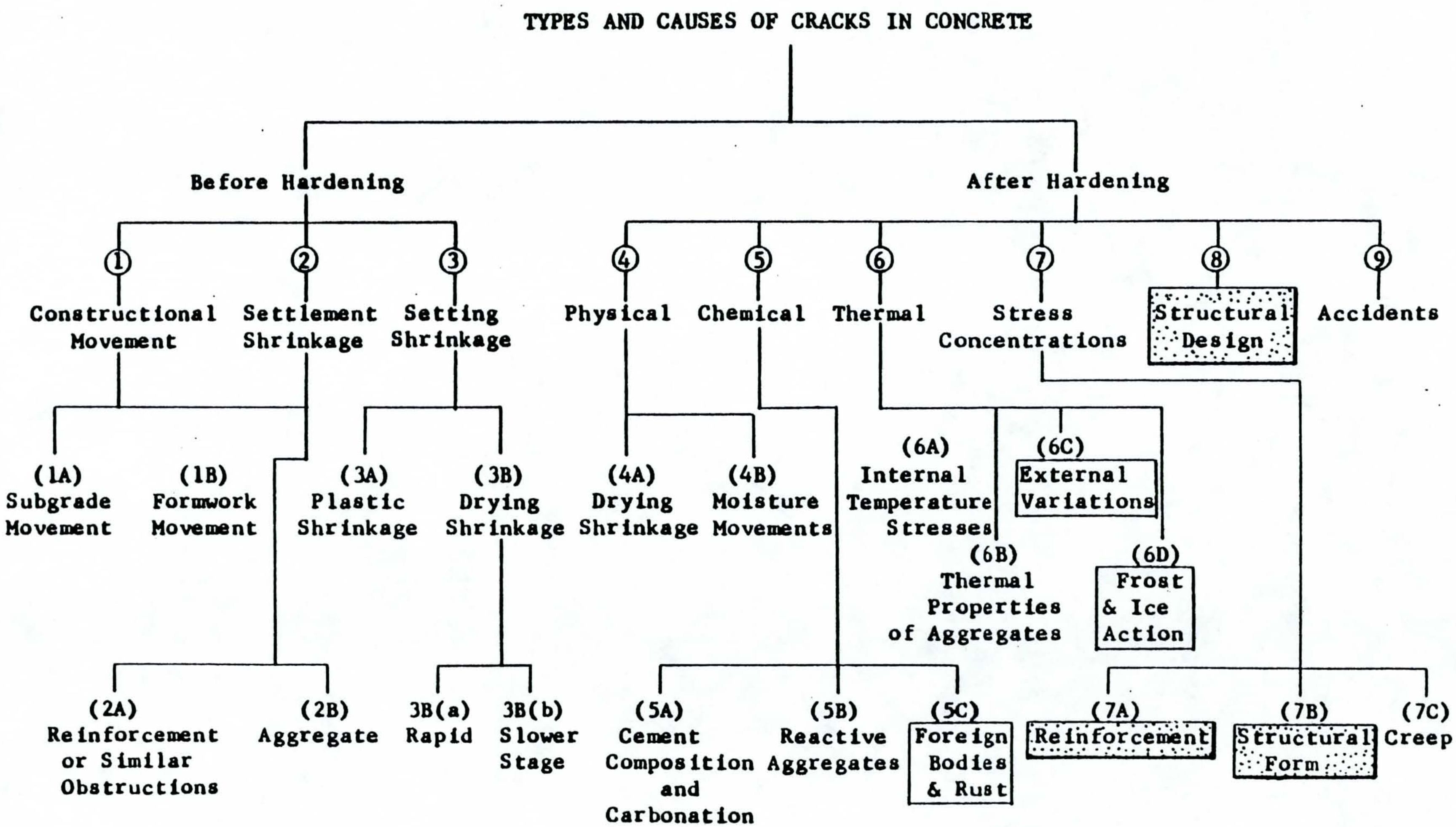
In trapezoidal channels restraint to slab movement is caused by location of cutoff walls, certain types of joints, and the weight of concrete.

Rectangular channels are restrained by lateral soil pressure on the sidewalls, the weight of soil on the heel of the wall, the weight of the sidewalls as transmitted to the invert of the channel as well as the weight of the invert, location of cutoff walls, and certain types of joints.

Short slabs of concrete need no reinforcement since plain concrete is able to develop some tensile strength to resist minor dimensional changes. Longer slabs will require reinforcement in proportion to the slab length because the restraint to movement as a result of friction between slab and subgrade increases with slab length and will exceed the tensile strength of the concrete.

The basis of the design criteria is the principle that "the amount of reinforcing required to reduce the size and spacing of concrete cracks is a function of the restraint imposed on the slab movement due to temperature variation."

Development of this principle by means of simple static analysis results in the subgrade drag equation and in its common form (with modified definitions) is:



After Permanente Cement Company

Figure A-1
TYPES AND CAUSES OF CRACKS IN CONCRETE

$$A_s = \frac{CLW}{2f_s}$$

where

A_s = effective cross-sectional area of steel in square inches per foot length (or width) of section

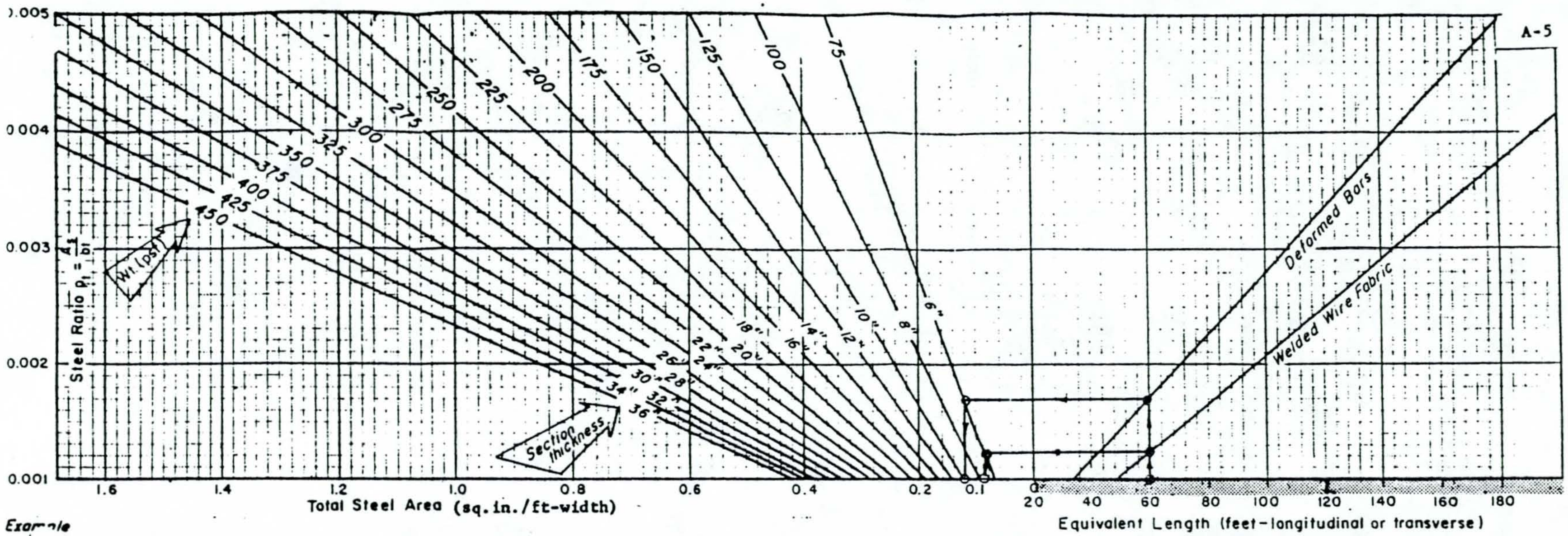
C = coefficient of subgrade friction

L = distance in feet between unrestrained ends of slab. (If the ends are restrained, an equivalent length should be used.)

W = weight of the slab in pounds per square foot. (An equivalent weight of slab should be used for rectangular channels.)

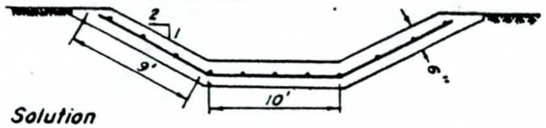
f_s = allowable unit tensile stress in the reinforcing steel in pounds per square inch.

This equation with appropriate values for constants C and f_s has been plotted in graph form in Figure A-2.



Example

TRAPEZOIDAL CHANNELS



Given: R/C Trapezoidal lining 6" thick

Find: Required reinforcing steel with joints at 30' spacing:
A. No cutoffs
B. With cutoffs

Solution

A. No cutoffs

(1) Transverse Reinforcement
Channel perimeter length = 2(9) + 10 = 28'
A_s required minimum (p_1 = 0.001)
a. Deformed bars A_s = 0.07 in.^{2} use #3 @ 18" A_s = 0.08 in.^{2}
b. Welded wire fabric A_s = 0.07 in.^{2} use 66-44 A_s = 0.08 in.^{2}

(2) Longitudinal Reinforcement
Joint spacing at 30'
A_s required minimum (p_1 = 0.001) See (1) above

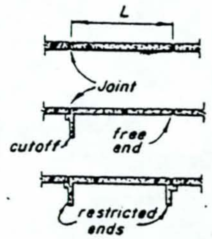
B. Cutoff at upstream end of slab

(1) Transverse Reinforcement See A-(1) above

(2) Longitudinal Reinforcement
Equivalent joint spacing (2L) = 60'
a. Deformed bars A_s = 0.12 in.^{2} #4 @ 18" A_s = 0.12 in.^{2}
#3 @ 11" A_s = 0.13 in.^{2}
b. Welded wire fabric A_s = 0.09 in.^{2} 44-44 A_s = 0.12 in.^{2}

RECTANGULAR CHANNELS

See Appendix for procedures in determining equivalent weights used in proportioning longitudinal reinforcement for rectangular channels.



DISTANCE BETWEEN JOINTS	END RESTRICTION	*EQUIVALENT LENGTH
L	Both ends free	L
L	One end restrained	2L
L	Both ends restrained	3L
L	Cutoff in Center	L

*Equivalent length depends on end restraint.

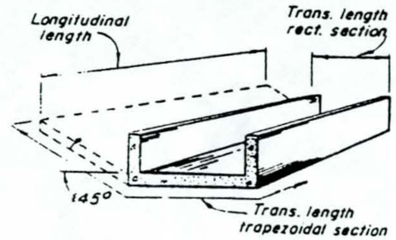


FIGURE A-2

SHRINKAGE AND TEMPERATURE REINFORCING STEEL

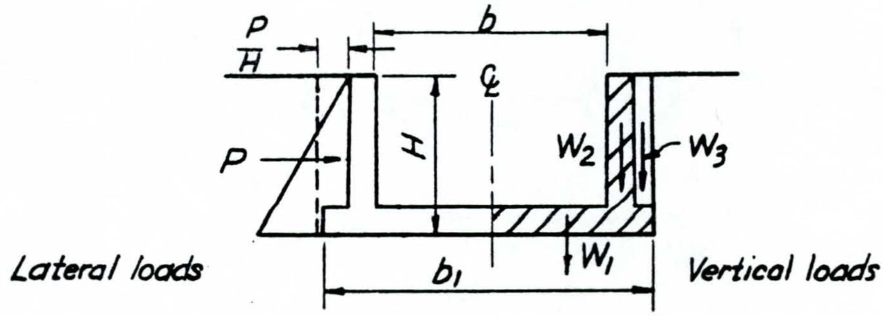
Channel lining - Paving slab - Retaining walls (Long. bars)

December 1966

Procedure for Determining Shrinkage and Temperature Steel Requirements Using the Modified Subgrade Drag Principle

A. Longitudinal Direction

1. R/C Rectangular Channel Lining



a. Sidewalls

(1) Determine P (total EFP on the sidewall).

(2) Convert to equivalent uniform load $W_e = \frac{P}{H}$

(3) Enter Figure A-2 with appropriate joint spacing and reinforcement type to determine steel area per 12-inch width.

(a) For uniform wall thickness use uniform steel quantity for entire height.

(b) For vertically tapered walls vary steel area, normally in two steps.

b. Floor slab (width $b < 3H$ no separate paving slab)

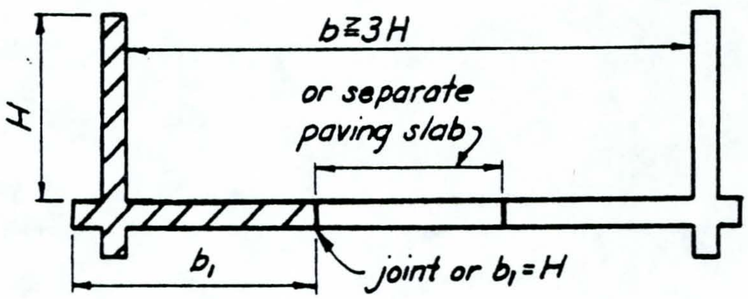
(1) Calculate vertical load $\Sigma W = W_1 + W_2 + W_3$

(2) Convert to equivalent uniform load

$$W_e = \frac{2\ \Sigma W}{b_1}$$

(3) Enter Figure A-2 with appropriate joint spacing (same spacing as in sidewall) and reinforcement type to determine steel area per 12-inch width.

c. Floor slabs (widths b $\geqq$ 3H)



(1) Calculate total vertical load for sidewall, heel load, and invert slab to joint or distance $b_1 = H$.

(2) Calculate total vertical load (see b).

(3) Convert to equivalent uniform load $W_e = \frac{\Sigma W}{b_1}$

(4) Enter Figure A-2 with appropriate joint spacing (same spacing as in sidewall) and reinforcement type to determine steel area per 12-inch width.

(5) Determine steel area in remainder of invert by entering chart, Figure A-2, with thickness or weight of paving slab and appropriate joint spacing.

2. R/C Trapezoidal Channel Lining

Enter chart, Figure A-2, with section thickness and determine steel per foot of lining for appropriate joint spacing.

B. Transverse Direction

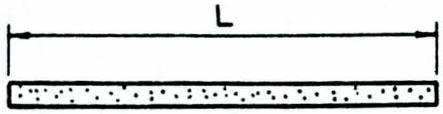
1. R/C rectangular section $p_t = 0.0025$ or 0.003 for single or double grid slabs, respectively.

2. R/C trapezoidal channel lining.

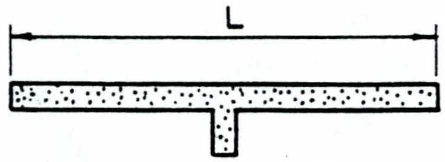
   Enter chart, Figure A-2, with section thickness and length of perimeter or distance between contraction or expansion joints, whichever is less, and determine steel area required.

C. Equivalent Slab Length

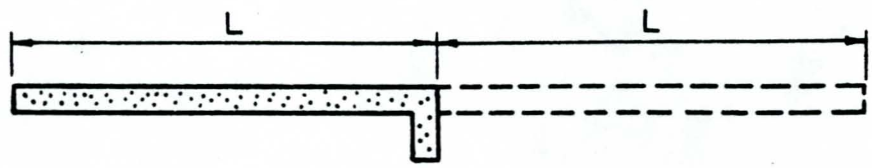
Where a cutoff wall is placed monolithic or anchored with the lining, its location is important in determining equivalent slab length. The following diagrams should be self-explanatory.



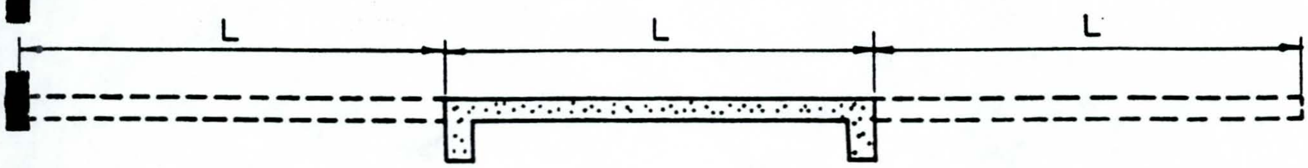
Equivalent length $\ell = 1L$
Both ends free to shrink toward center.



Equivalent length $\ell = 1L$
No end restraint. Cutoff located at point of no shrinkage movement.



Equivalent length $\ell = 2L$
Cutoff restricts movement at one end. All movement must come from free end. The cutoff so located acts as though it is located in the center of a longer slab.



Equivalent length $\ell = 3L$
End restraint at both ends. This section will require 3 times the steel required in the first two sketches for the same joint spacing.

ALBUQUERQUE METROPOLITAN ARROGO FLOOD CONTROL AUTHORITY

DESIGN GUIDE

FOR

TRAPEZOIDAL CONCRETE FLOOD CONTROL CHANNELS

City of Albuquerque
and
Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA)

DRAFT - JULY 7, 1981
(With 4/2/82 Revisions)
and 5/13/82 Revisions

Flood Control channels are to be designed in accordance with the standards outlined herein. These standards represent compromises among many considerations including first cost, right-of-way, safety and maintenance. When compliance is not possible, alternate design concepts may be proposed.

This supersedes Section II of AMAFCA STORMWATER CHANNEL DESIGN STANDARDS dated 4/14/80.

It is anticipated that the Design Guide will be modified in the future. Comments and suggested changes are encouraged.

DESIGN GUIDE

FOR

TRAPEZOIDAL CONCRETE FLOOD CONTROL CHANNELS

I. GENERAL

The criteria included herein are intended to provide standards for the design and construction of trapezoidal concrete lined channels in the Albuquerque Metropolitan Area. Alternates to these standards may be proposed where unique or special conditions are encountered.

II. DESIGN MEMORANDUM

Before proceeding to preliminary design, a Design Memorandum should be prepared and approved by Public Authority. The purpose of the Design Memorandum is to describe design assumptions and concepts, and serve as a future reference. Properly used, the Design Memorandum can avert false starts and wasted effort.

The following are among items which should be addressed in the Design Memorandum:

- o Description of the project.
- o Peak flow rate and hydrology on which it is based.
- o Water velocities by channel reach.
- o Profiles of channel invert and top of concrete for lowest bank and any superelevated bank.
- o Profiles of critical depth, normal depth, energy grade line, and design water surface based on backwater or forewater calculations.

- o Upstream collection, downstream energy dissipation, and other channel transitions. Transitions require backwater or forewater analysis with cross section spacing not to exceed 10 feet.
- o Bridges or other traffic crossing structures.
- o Channel junctions and inlets with hydraulic parameters, e.g. angle of intersection, velocities, water surface elevations and Freude numbers for full range of flow possibilities.
- o Proposed deviations from this Guide and justification therefor.
- o Any unusual feature of the project.
- o Concrete reinforcing and joint design.
- o Maintenance access, schedule, and budget estimate.
- o Right of way available or to be dedicated.
- o Relationship to upstream and downstream facilities, both in existence and planned.

III. DESIGN STORM

A 100-year frequency storm shall be used unless otherwise specified by Public Authority.

IV. DRAINAGE WAY

A. Right-of-Way

Sufficient right-of-way (fee simple or easement) shall be provided to accommodate the channel along with the necessary channel appurtences. Temporary construction easements may be required.

B. Maintenance Road

A maintenance road having a minimum width of 15 feet shall be provided on at least one side of the concrete lined channel.

Turnouts sufficient to allow passages of two 5-ton trucks shall be provided at no more than 1/2 mile intervals and turnarounds shall be provided at access road dead ends.

Ingress and egress shall be provided from public right-of-way to the channel maintenance road.

C. Fencing

Chain-link fence shall be installed to deny access wherever vertical walls exceed 4 feet.

V. EARTHWORK

The following shall be compacted to at least 90% of maximum density as determined by ASTM D-1557 (modified Proctor):

A. The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).

B. Top 12 inches of maintenance road.

C. Top 12 inches of earth surface within 10 feet of concrete channel lip.

D. All fill material.

VI. CONCRETE

A. Materials

(1) Cement type: ILA or I-IILA

(2) Minimum cement content: 5.5 sacks/c.y.

(3) Maximum water-cement ratio: 0.53 (6 gals. per sack)

(4) Maximum aggregate size: 1½ inches

(5) Air content range: 4-7%

(6) Maximum slump: 3 inches

(7) Minimum compressive strength (f'_c): 3000 psi @ 28 days

(8) Class F Flyash meeting the requirements of ASTM C 618 shall be proportioned in the mix at a 1:4 ratio of flyash to cement weight.

(9) Steel reinforcement shall be grade 60 deformed bars. Wire mesh shall not be used.

B. Lining Section

(1) Bottom width - 10 feet minimum

(2) Side slopes - 1 vertical to 2 horizontal maximum slope

(3) Concrete lining thickness

Channel side slopes:	Minimum 6 inches
Channel bottom:	Minimum 7 inches, thicken to 8 inches when design velocity exceeds 25 feet per second

(4) Concrete finish

The surface of the concrete lining shall be provided wth a wood float finish. Precautions shall be taken to guard against excessive working or wetting of finish.

(5) Concrete curing

All concrete shall be cured by the application of liquid membrane-forming curing compound (white pigmented) immediately upon completion of the concrete finish.

(6) Ladder-type steps shall be installed not farther than 700 ft. apart on alternating sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above channel invert.

C. Joints

(1) Insofar as feasible, channels shall be continuously reinforced without transverse joints. However, expansion joints may be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

(2) The preferred design avoids longitudinal joints. However, if included, longitudinal joints should be on side slope at least one foot vertically above channel invert.

(3) All joints shall be designed to prevent differential displacement.

(4) Construction joints are normally appropriate at the end of a day's run, where lining thickness changes, and any time concrete placement stops for more than 45 minutes.

D. Reinforcing Steel for Continuously Reinforced Channels

(1) Ratio of longitudinal steel area to concrete area (Asl) $\geq$ .005.

(2) Ratio of transverse steel area to concrete area (Asr) $\geq$ .0025.

(3) Steel placement: Near center of section; however, minimum clear cover over eath is 3 inches (NOTE: Inspectors must ensure this requirement is not violated by contractors during pouring operations.)

VII. HYDRAULICS

A. Freeboard

Adequate channel freeboard above the designed water surface shall be provided and shall not be less than determined by the following:

$$\text{Vertical freeboard (Feet)} = 2.0 + 0.025\, v \sqrt[3]{d}.$$

Freeboard shall be in addition to superelevation, standing waves and/or other water surface disturbances. These special situations should be addressed in the Design Memorandum.

Normally, concrete side slopes should be extended to provide freeboard. However, additional freeboard to compensate for uncertainties may be provided by vertical splashboards adequately connected to the side slopes.

B. Superelevation

Superelevation of the water surface shall be determined at all horizontal curves and design of the channel section adjusted accordingly.

Superelevation shall be determined by the following equation:

$$\Delta y = \frac{V^2 W}{gr}$$

where

Δy = rise in water surface between a theoretical level water surface at the center line and outside water-surface elevation (superelevation)

V = mean channel velocity

W = channel width at elevation of center-line water surface

g = acceleration of gravity

r = radius of channel center-line curvature

If the total rise in water surface (superelevation plus surface disturbances) is less than 0.5 ft., the calculated channel freeboard is adequate. No special treatment such as increased wall heights or spiral transitions is required.

Full superelevation shall occur over the entire outside bank of the curve and for a minimum 100 ft. downstream.

C. Freude Number

Except at designed transitions, Freude Number shall be either less than 0.7 or greater than 1.3.

D. Cross Sections

Design drawings shall display cross sections for concrete channel and adjacent land. Cross section intervals shall be displayed wherever topography changes significantly but at least every 100 ft.

VIII. TRANSITIONS

Transitions in the concrete channel shall be held to a minimum. Where transitions are necessary they shall be designed to minimize flow disturbance.

A. Earth Channel to Concrete Lining Transition

The mouth of the transition shall match the earth channel section as closely as practicable. Wing dikes and/or other structures shall be provided to positively direct all flows to the transition entrance.

The upstream end of the concrete lined transition shall be provided with a cutoff wall having a minimum depth of 4 feet and extending the full width of the concrete section. Erosion protection directly upstream of the concrete transition consisting of derrick stone, wire-tied rock riprap or equivalent at least 30 feet in length and extending full width of the channel section shall be provided. Wire-tied rock riprap shall be a minimum of 12 inches thick and tied to the concrete lining and cutoff wall.

B. Concrete Lining to Earth Channel Transition

The transition from concrete lined channels to earth channels shall include an energy dissipator as necessary to release the designed flows to the earth channel at approximately natural conditions.

Since energy dissipator structures are dependent on individual site and hydraulic conditions, detailed criteria for their design has been purposely excluded and only minimum requirements are included herein for the concrete to earth channel transition.

On this basis the following minimum standards shall govern the design of concrete to earth channel transitions:

(1) Maximum rate of bottom width transition:

Max. Water Velocity	
0-15 fps	1:10
16-30 fps	1:15
31-40 fps	1:20

(2) The downstream end of the concrete transition structure shall be provided with a cutoff wall having a minimum depth of 4 feet and extending the full width of the concrete section.

(3) Directly downstream of the concrete transition structure erosion protection consisting of derrick stone, wire-tied rock riprap or equivalent shall be provided. Erosion protection must extend at least 50 feet downstream. Additionally, it must contain any hydraulic jump, both vertically and horizontally. Wire-tied rock riprap shall be a minimum of 12 inches thick and tied to the concrete structure and the cutoff wall.

C. Vertical Transitions

All changes in longitudinal channel slope shall be gradual using a vertical curve or short tangential segments. Changes greater than 4 degrees shall include spiral transitions. Special analysis is required for design velocities greater than 25 fps.

D. Transitions at Street Crossing Structures

It is preferred that the channel section be continuous through crossing structures. However, when this is not practicable, channel disturbance shall be minimized.

IX. INLETS

Local and exterior drainage arising more than 20 ft. from the edge of the concrete channel shall be allowed to enter the channel through designed inlets only. Inlets may consist of the following types:

A. Surface Inlet

Surface-type inlets shall be constructed of concrete having a minimum thickness of 6 inches and shall be reinforced with the same steel as 6" concrete lining. The upstream end of the surface inlet shall be provided with a concrete cutoff wall having a minimum depth of three feet and the downstream end of the inlet shall be connected to the channel lining by an expansion joint. Side slopes of a surface inlet shall be constructed at slopes no greater than 1 vertical to 10 horizontal to allow vehicular passage across the inlet.

Drainage ditches or swales immediately upstream of a surface inlet shall be provided with erosion protection consisting of concrete lining, rock riprap or other non-erosive material.

Surface inlets shall enter the channel at a maximum of 90^{o} to the channel centerline, i.e., they may not point upstream.

B. Pipe and Box Culvert Inlets

Additional design criteria is outlined in Corps of Engineers Manual EM 1110-2-1601, Paragraph 18 (h).

Pipes and box culvert penetration of channel linings shall be isolated by expansion joints. Continuously reinforced channels shall be designed to accommodate extra stress at these discontinuities.

C. Side Channels

Side channels shall be designed to match main channel flow characteristics as closely as feasible. Design and construction shall be similar to the main channel. Additional criteria is provided in Corps of Engineers Manual EM 1110-2-1601, Paragraph 18(d).

X. CHANNEL ACCESS

Channel access ramps for vehicular use shall be provided as necessary for complete access to the channel throughout its entire length with the maximum length of channel between ramps being one-half mile.

Ramps shall be constructed of 8" thick reinforced concrete and shall not have slopes greater than 17%. Ramps shall not enter the channel at angles greater than 15° from a line parallel to the channel centerline. Ramps shall be constructed on the same side of the channel as the maintenance road.

The maintenance road shall be offset around the ramp to provide for continuity of the road full length of the channel.

The downhill direction of the ramp shall be oriented downstream.

Channel capacity shall not be reduced.

XI. STREET CROSSINGS

Street crossing or other drainage structures over a concrete-lined channel shall be of the all-weather type, i.e., bridges or concrete box culverts. Bridges are preferred over box culverts. Rundowns from the street shall be incorporated in the design.

Channel lining transitions at bridges and box culverts shall conform to the provisions for transitions listed elsewhere in this Guide.

Drainage structures having a minimum clear height of 8 feet and being of sufficient width to pass maintenance vehicles may result in minimizing the number of required channel access ramps.

A. Bridges

Bridges shall be designed with the minimum number of spans practicable. Piers shall have continuous webbing and be streamlined both up and downstream (see Corps of Engineers Manual EM 1110-2-1601, Paragraph 9).

Concrete channel lining, including reinforcing, shall be continuous under bridges without contraction or expansion joints. Construction joints shall be provided as necessary.

Expansion joint materials 3/4" thick with sealant shall be provided around all lining penetrations, such as bridge piers. In addition, expansion joint material 3/4" thick with sealant shall be provided where the concrete lining abuts bridge abutments.

B. Box Culverts

Box culverts shall be designed having the minimum number of openings practicable. Minimum height of box culvert openings shall be 4 feet.

Concrete channel lining shall be connected directly to the concrete box culvert complete with 100% of lining reinforcement and keyed construction joint. No expansion joints shall be installed. Construction of new box culverts shall include the projection of reinforcing steel from the end face of the box culvert to match lining reinforcement.

Lining connections to existing box culvert shall include the drilling of holes and grouting in of projecting reinforcement in the end face of existing structures to match channel lining reinforcement.

VENTURA COUNTY FLOOD
CONTROL DISTRICT

VENTURA COUNTY FLOOD CONTROL DISTRICT

DESIGN MANUAL

ADOPTED BY THE BOARD OF SUPERVISORS

VENTURA COUNTY FLOOD CONTROL DISTRICT

A. P. STOKES, ENGINEER-MANAGER

J. B. QUINN, DEPUTY ENGINEER-MANAGER

JULY, 1968

450. TRAPEZOIDAL OPEN CHANNELS

451. GENERAL

The use of trapezoidal channels shall be discussed with the District before detailed designs are started. Since trapezoidal channels require considerably more right of way, extensive maintenance, and cannot be readily covered, the use of trapezoidal channels in Flood Control projects is not favored.

452. REINFORCED CONCRETE TRAPEZOIDAL CHANNELS

.10 *METHOD OF DESIGN*

All reinforced concrete trapezoidal channels shall be lined with cast-in-place concrete and shall consist of an invert slab, side slope lining and a beam section at the top of the side slopes. Air blown mortar is permitted only for warped transition walls. A minimum acceptable channel section is shown on Standard Drawing No. 12.

.20 *DESIGN LOADS*

Horizontal and vertical loads used in the design of trapezoidal open channels shall be in accordance with the following criteria:

.21 *Horizontal Loads* - Channel walls shall be designed to sustain the entire lateral pressure in shear and 50% of the resultant moment at the base of the side slopes. The lateral pressure shall be computed on the vertical projection of the side slope walls. The lateral pressure shall include an equivalent fluid pressure of not less than 40 psf, live load and loads due to sloping surcharge. Except when located adjacent to railroad tracks, the live load shall consist of a minimum of 2 feet of surcharge distributed uniformly on the grade adjacent to the side slope walls. Surcharge due to railroad shall be determined using E72 loading applied adjacent to the walls. Surcharge due to sloping backfill may be computed from Figure 423.11(a).

.22 *Vertical Loads* - Soil and uplift pressures on the invert slab shall be computed considering it as a slab with uniform load applied to it. In designing the invert slab, consideration shall be given to the weight of the entire structure, however, in computing the uplift forces the weight of the invert slab alone shall be used as a resisting force. See Section 470, "Subdrain Systems".

.30 *STRUCTURAL SECTION*

See Standard Drawing No. 12 for typical section.

.31 *Thickness of Members* - Side slope wall and invert slab shall have a minimum thickness of 6 inches, measured normal to the face of the concrete. For velocity in excess of 25 fps, the thickness of the members shall be increased to 7 inches.

.32 *Reinforcing* - Reinforcing steel in side slope walls and in the invert shall consist of a minimum of #4 bars at 12 inch centers. However, additional reinforcing shall be provided as necessary by the design. A minimum lap of 20 and 30 bar diameters shall be provided in all splices, in longitudinal and transverse reinforcing steel respectively. However, the length of the lap shall not be less than 12 inches.

.33 *Steel Clearance* - Steel clearance shall be shown on project drawings from the edge of the bars to the face of the concrete. Clearance on the earth side shall be a minimum of 3 inches, and not less than 2 inches on the inside face.

.34 *Transverse Slope of Invert Slab* - The drop in the invert slab from the toe of the side slope walls to the center of the channel shall be based on the following criteria:

b = 0' - 10'	Flat
b = 11' - 20'	Drop = 2"
b = 21' or greater	Drop = 3"

.40 *JOINTS*

.41 *Longitudinal Joints*- Longitudinal keyed construction joints shall be provided at the junction of the side slope walls and the invert slab. The keyed joints shall be 1½ inches deep for the entire thickness of the side slope walls. Transverse steel shall be continuous through the construction joints.

.42 *Transverse Joints*

a. Contraction Joints - Transverse contraction joints shall be spaced at 20 foot intervals in the side slope walls and the invert. Contraction joint shall be 3/8 inch wide for 1/3 of the depth of the member. Longitudinal reinforcing shall extend through the transverse contraction joints. The spacing of transverse contraction joints may be reduced to eliminate proximity to junction structures. Minimum spacing shall be 10 feet.

b. Construction Joints - Transverse construction joints shall be in the same plane in the wall and invert slabs. Longitudinal steel shall be discontinued at the transverse construction joint.

.50 *SUBDRAINAGE*

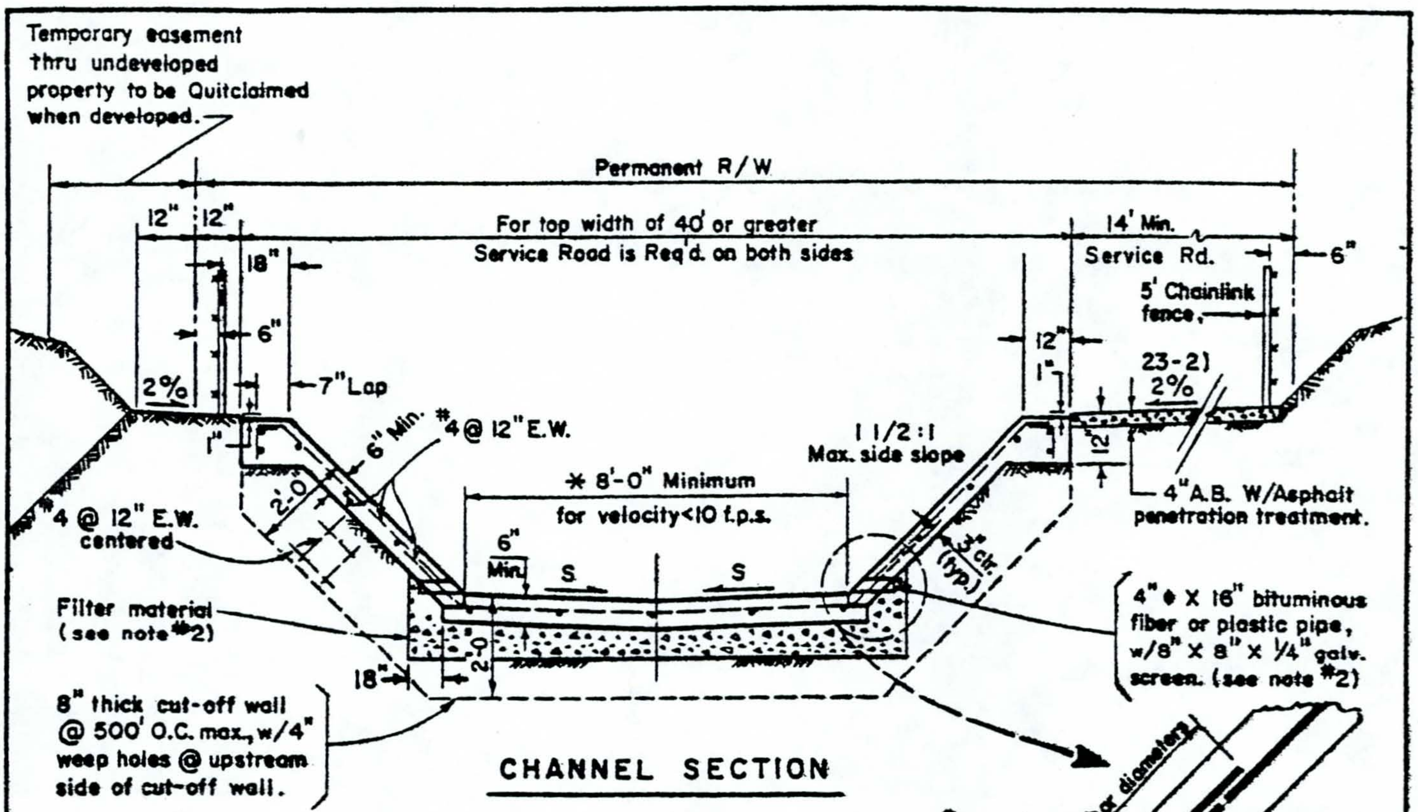
Reinforced concrete trapezoidal channels are inherently weak against uplift pressures, and require careful investigation of ground water conditions and the need for subdrains. See Section 470, "Subdrain Systems", for discussion of types of subdrains.

453. RIP RAP SLOPE PROTECTION FOR TRAPEZOIDAL CHANNELS

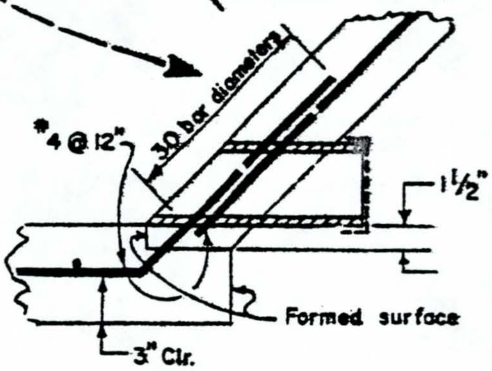
.10 *GENERAL*

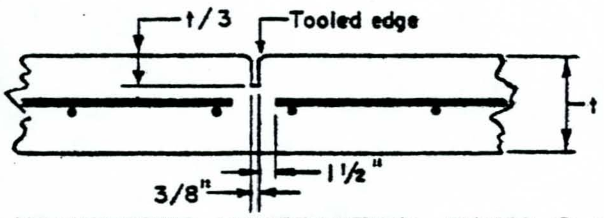
Various methods of providing rock rip rap protection on the side slopes of trapezoidal channels are in use. However, the methods acceptable by the District for use as temporary slope protection are (1) dumped rock rip rap, (2) grouted rip rap and (3) sacked concrete rip rap. The term rip rap as used herein is defined as "a layer, facing or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used"*. The latter of the two protective methods described above shall generally be used for temporary protection at inlets and outlets of permanently improved conveyance structures. Dumped rip rap slope protection may be used as ultimate improvement for fairly wide major channels on a relatively flat grade. District approval shall be obtained prior to use of rock rip rap trapezoidal

* *Reference 15*



CHANNEL SECTION





Bot. width	S.
0 - 10'	"
11' - 20'	2"
21' - ___	3"

TRANSVERSE CONTRACTION JOINTS @ 20' O.C.

NOTES:

1. In undeveloped areas and/or where significant sheet flow is intercepted by the channel, a system of parallel ditches and side drain inlets approved by the Flood Control District shall be constructed to control surface flow.
2. Filter material design and weep hole spacing requirements to be determined by soils test of native material.
3. An adequate longitudinal pipe subdrain system of approved design shall be provided where required by high groundwater conditions.
4. Reinforced concrete structural section shown is minimum allowable. The connection between side walls and bottom slab shall be designed for shear and 50% of the moment based on a vertical projected area and equivalent fluid pressure of not less than 40 pounds per cubic foot.
5. Subgrade for lining shall be firm undisturbed natural ground or fill compacted to 90% relative compaction for full depth of fill in accordance with U.B.C. 70-1.
6. See Section 450 of the Design Manual for detailed discussion of the design of trapezoidal R.C. Channels.
7. The use of trapezoidal design in lieu of rectangular must be approved in each case by the District for channel under the jurisdiction of the District.

Ventura County Flood Control District DESIGN STANDARDS
R.C. TRAPEZOIDAL CHANNEL SECTION MINIMUM DESIGN STANDARDS
Approved A P Stoker Date 5.14.68

ACI AND PCA

concrete INFORMATION

pca

Design of Concrete Pavement for City Streets

Standards established by a community for the design and construction of its streets should provide for pavements with long service life and low maintenance. Excess maintenance of inadequate pavements (such as patching chuckholes and applying periodic seal coats) is an unnecessary drain on tax dollars. An investment in adequate concrete streets needing little maintenance over a long service life—50 years or more in many communities—frees more dollars for permanent capital improvements.

Concrete pavements are designed for both economy and long service. Following are the factors involved in designing concrete pavement for the lowest possible annual cost:

1. Street classification and traffic (including axle weights and volume)
2. Thickness design
3. Design life
4. Concrete quality
5. Subgrade strength and character
6. Geometric design
7. Jointing
8. Construction specifications

STREET CLASSIFICATION AND TRAFFIC

Comprehensive traffic studies made within city boundaries can supply necessary data for the design of municipal pavements. A practical approach is to establish a street classification system. Streets of similar character have essentially the same traffic densities and axle load intensities. The following street classifications are used in this information sheet:

Light Residential Streets. These streets are not long and in subdivisions they may be dead end with a turnaround. Light residential streets serve traffic to and from only a few houses or lots (20 to 30). Traffic volumes are low, less than 200 vehicles per day (vpd) with 1% to 2% heavy commercial traffic (2-axle, 6-tire trucks and heavier). Trucks using these streets will have a maximum tandem axle load of 36 kips and a 20-kip-maximum single-axle load.

Residential Streets. In subdivisions these streets carry the same type of traffic as light residentials, but serve more houses (60 to 140) including those on dead-end streets. In cities with a grid-type street pattern, traffic generally consists of vehicles serving the homes plus an occasional heavy truck. Traffic volumes range from 300 to 700 vpd with 1% to 2% heavy commercial traffic per day (hcvpd).

Residential Collector Streets. Residential collectors receive all the residential street traffic within an area and distribute it into the major street systems. They can be quite long, serving 140 to 300 homes or more, and have traffic volumes of 700 to 1,500 vpd with 1% to 2% heavy commercial traffic.

Collector Streets. Collectors may serve several subdivisions and may be several miles long. They may be bus routes and serve truck movements to and from an area although they are not through routes. Traffic volumes vary from 2,000 to 6,000 vpd with 3% to 5% heavy commercial traffic. Trucks using these streets have a maximum tandem axle load of 38 kips and a 24-kip-maximum, single-axle load.

Arterial Streets. Arterials bring traffic to and from expressways and serve major movements of traffic within and through metropolitan areas not served by expressways. Truck and bus routes as well as state and federal numbered routes are usually on arterials. For design purposes, arterials are divided into minor arterials, arterials, and major arterials depending on traffic capacity and type. A minor arterial may have fewer travel lanes and carry less volume of total traffic, but the percentage of heavy trucks may be greater than that on a 6-lane major arterial.

Expressways. Expressways are designed to move large volumes of traffic at relatively high speeds. Extensive and thorough designs are justified and are not included here.

Business Streets. These streets are a special category. They provide land access to stores and at the same time serve traffic in the central business district. Business streets are frequently congested. Traffic speeds are low. Traffic volumes, however, are relatively high with a low percentage of truck traffic.

Industrial Streets. Industrial streets provide access to industrial areas or parks. Total traffic volume may be in the lower range but the percentage of heavy trucks with heavy axle weights is high.

The street classifications outlined here may or may not correspond to the classifications used in any metropolitan area. They are given here to indicate in a general way the volumes and axle weights of traffic using streets. They are summarized in Table 1. The values are reasonable but should be tempered with knowledge of local traffic patterns.

Table 1. Street Classifications and Normal Concrete Pavement Thicknesses

Street classification	Vpd or ADT 2-way	Lots, No.	Heavy commercial vehicles, 2-axle, 6-tire and heavier		Normal concrete pavement thickness, inches	Maximum axle load, kips	
			Percent	No. per day		Tandem	Single
Light residential	200	20-30	1-2	3-5	5-6	36	20
Residential	300-700	60-140	1-2	5-11	5-6	36	20
Residential collector	700-1,500	140-300	1-2	11-23	6-7	36	20
Collector	2,000-6,000		3-5	80-240	6-7	38	24
Minor arterial	3,000-7,000		10	300-700	7	46	35
Arterial	6,000-13,000		5-7	360-780	8	56	30
Major arterial	14,000-28,000		5	700-1,400	8-9	65	40
Business	11,000-17,000		3-5	440-680	8	56	30
Industrial	2,000-4,000		15-20	350-700	9	65	40

THICKNESS DESIGN

The number and weights of heavy axle loads expected over the design life of a street together with concrete flexural strength and subgrade support are needed for its complete design. Three design methods are outlined.

Design Method 1

Use Table 1 for the range of concrete thickness normally built for each class of street.

Design Method 2

A series of six design graphs are given at the end of this information sheet. They were developed for the street classifications as follows:

Charts 1 and 2 for light residential, residential, and residential collectors
Chart 3 for collectors
Chart 4 for minor arterials
Chart 5 for arterial and business
Chart 6 for major arterial and industrial

With the appropriate chart, proceed as follows:

1. Decide if the maximum axle load weights shown in Table 1 confirm local knowledge. Table 1 values are reasonable but probably heavier than those normally anticipated.[1,2]*
2. Decide on the design life for the street.
3. Estimate the average two-way heavy commercial vehicles per day that will use the facility during the design life. If no data are available, heavy truck traffic counts should be made. If a count is not made, the traffic data in Table 1 can be used as a guide. An alternative for residential streets is to estimate the number of lots or houses to be served by the street.
4. The 28-day modulus of rupture in flexure (MR) of concrete is usually used in design. See the discussion under "Concrete Quality."
5. The supporting power of the subgrade is expressed as a k value. The modulus of subgrade reaction can be determined by plate bearing tests, estimated from correlation tests, or obtained from the guides given in the section "Subgrade Strength and Character."
6. Enter the traffic chart from the left (hcvpd), project horizontal line to the MR line, then vertically to k-value line, and horizontally to design thickness at the right. (The broken line on each chart is an example.)

Design Method 3

The third method is a complete design analysis recommended for larger municipalities with adequate data on traffic, materials, and soils. See Portland Cement Association's *Thickness Design for Concrete Pavements,*[3] particularly the modified PCA method.

*Superscript numbers refer to references at the end of this information sheet.

In the modified method, truck traffic counts are taken and separated into vehicle classifications. This is especially important since the percentage of tractor semitrailer and combination trucks may be quite different on some types of streets from that reported in a typical W-4 loadometer table. Highway departments in cooperation with the Federal Highway Administration conduct loadometer and traffic volume studies, results of which are compiled in a set of tables coded W-1 through W-8. The W-4 tables tabulate the number of axle loads by weight group for each type of truck. Using the urban section of these studies, it is possible to make a simplified truck classification count and then distribute the axle loads for each truck type in accordance with the weight distribution pattern found in the W-4 table.

The heaviest axle loads control concrete pavement thickness design but these loads, which are given in the W-4 table, often occur only on a few routes. Therefore, if the data are being used for load distributions in an area divorced from these few routes, it is feasible to exclude the unrelated load groups when using the W-4 load data.

On major routes with multiple lanes, the distribution of commercial vehicles in each lane must be considered. A reasonable assumption for streets with two lanes in each direction is that 85% to 90% of the commercial vehicles will be in the right-hand lane.

DESIGN LIFE

Given known traffic, a concrete pavement can be designed for any life desired; however, future changes in traffic patterns and weights are often difficult to predict. For heavily traveled roads and streets, future traffic can be of considerable influence on design. On the other hand, future changes in traffic for residential and other lightly traveled streets are often of little significance. Fifty years is commonly used in designing pavements, especially for streets classified as residential, since residential streets are rarely subject to relocation or realignment. A design life of 35 and 50 years has been used for the designs presented here.

CONCRETE QUALITY

Concrete paving mixes are designed (1) to give satisfactory durability under the conditions the pavement will be subjected to and (2) to produce the desired flexural strength.

Since the critical stresses in concrete pavements are flexural rather than compressive, the flexural strength of concrete (expressed as MR) is used in concrete pavement design. Under average conditions concrete that has an MR (ASTM C78, third-point loading) of 550 to 700 psi at 28 days is most economical.

In frost-affected areas, concrete pavements subjected to many cycles of freezing and thawing must be protected from the action of deicing salts.[4] It is essential that the mix have a low water-cement ratio, an adequate cement factor, sufficient quantities of entrained air, plus adequate curing and a period of air drying. The amounts of entrained air needed for weather-resistant concrete vary with the maximum-size aggregate. The following percentages are recommended:

Maximum-size aggregate, in.	Total air content, %**
1-1/2	5 ± 1
3/4, 1	6 ± 1
3/8, 1/2	7-1/2 ± 1

In addition to making the hardened concrete pavement weather resistant, recommended amounts of entrained air improve the concrete while it is still in a plastic state by—

1. preventing segregation
2. increasing workability
3. reducing bleeding
4. reducing water required for satisfactory workability

Because of these beneficial and essential effects in both plastic and hardened concrete, entrained air should be incorporated into the mix designs for all concrete pavements.

Mixing water also has a critical influence on the durability and weather resistance of hardened concrete. The least amount of mixing water with a given cement content to produce a plastic, workable mix will result in the greatest durability in the hardened concrete. Laboratory and field experience with air-entrained concrete shows that, for satisfactory pavement durability, the water-cement ratio should not exceed 0.53 and the cement factor should not be less than 520 lb. per cubic yard. In areas of frequent freeze-thaw and where applications of deicing agents are common, the water-cement ratio should not exceed 0.50 with a minimum cement factor of 560 lb. per cubic yard.

Additional information on mix design can be found in *Design and Control of Concrete Mixtures.*[5]

SUBGRADE STRENGTH AND CHARACTER

Because of its rigidity, concrete pavement has remarkable beam strength and load-carrying capacity. Thus, the pressures beneath the concrete pavement are very low and distributed over relatively large areas. This ability of concrete to distribute heavy loads makes it unnecessary to build up subgrade strength with thick layers of crushed stone or gravel. Therefore, economical concrete pavements that will give good performance can be built on most in-place soils.*

Subgrade soils should be of uniform material and density for satisfactory pavement performance. Soft spots that show up during construction should be excavated and recompacted with the same type of material found in the adjacent subgrade. Uniform support cannot be obtained merely by dumping extra granular material on the soft spot.

With a reasonably uniform subgrade, excessive shrink and swell of expansive soils are substantially reduced by adequate moisture and density controls during compaction. The compaction of expansive soils at 1 to 3 percentage points over optimum moisture as determined by the AASHTO standard method keeps volume changes at a mini-

*Detailed information on materials, designs, and compaction requirements is contained in *Subgrades and Subbases for Concrete Pavements.*[6]

**These air contents will provide an adequate entrained air void system in the concrete when the air-entraining admixture meets the requirements of ASTM C260, "Specifications for Air-Entraining Admixtures for Concrete."

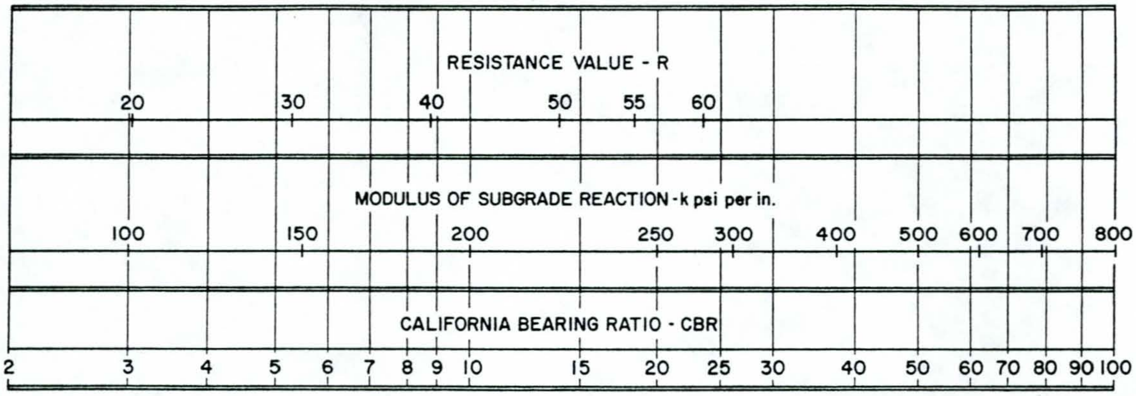


Fig. 1. Approximate interrelationships of soil bearing values.

mum. It also reduces differential frost action in northern climates. Special care must be taken in compacting the backfill of sewers, drainage facilities, and other permanent structures in the paved area.* The subgrade must not be allowed to dry before the pavement is constructed.

The supporting power of the subgrade is expressed as values of k, the modulus of subgrade reaction, and can be determined from plate bearing tests or by correlation with other soils of known k values.** For city street design, the following k values can generally be used:

k	Type of soil	Remarks
100	Silts and clays	Satisfactory
200	Sandy soils	Good
300	Sand-gravels	Excellent

While subbases beneath concrete pavements are not required for most city streets, pavements in the expressway or arterial classifications that carry large numbers of heavy trucks (more than about 100 to 200 two-way hcvpd) may require subbases to prevent mud-pumping of fine-grained subgrade soils. When subbases are required, they should be constructed with care.(6)

GEOMETRIC DESIGN

Utilities

Common practice in new subdivisions now dictates that utilities be placed in the right-of-way outside the pavement area to facilitate maintenance and additions to the utility systems. Present and future needs must be evaluated and provision made for them. Forethought can eliminate the tearing up of existing areas to increase drainage facilities.

Integral Curbs

A most practical and economical way to build concrete pavements for city streets is with an integral curb section. An integral curb is constructed with the pavement in a single operation—all concrete work being done simultaneously. The curb is easily formed with a templet and straightedge as the pavement is placed. Integral curbs can be constructed to almost any desired cross section.

Integral curb construction offers the designer an added safety factor due to the thickened edge section formed by the curb. Stresses and deflections at the pavement edge are reduced, thus increasing the structural capacity of the pavement. The inherent advantages and economy of integral curb construction recommend its consideration for city street pavements.

Street Widths

Street widths vary according to the traffic the street is designed to carry. The minimum recommended width, except in unusual cases, is 25 ft. with a maximum transverse slope of ¼ in. per foot of width. Consistent lane widths and cross slope are desirable.

Traffic lanes are customarily 10 to 12 ft. wide. Widths over 12 ft. are not recommended since experience has shown that drivers will attempt to pass on wider single lanes, promoting accidents.

Parking lanes along the curb are usually 7 to 8 ft. wide. A 7-ft. lane is used where passenger cars predominate, an 8-ft. lane where trucks must be accommodated. Parking lanes of 6 ft. are not recommended. On major streets parking lanes are 10 to 12 ft. wide and they can also be used as travel or turning lanes.

On streets where parking is prohibited, 2 ft. generally are provided along the curb as nontraveled space.

JOINTING

Joints must be carefully designed and constructed to ensure good performance. Except for construction joints, which divide paving work into convenient increments, joints in concrete pavements are used to keep stresses within safe limits and to prevent formation of irregular cracks. Suggested joint details for residential streets are given in *Integral Curb Pavement, Typical Section and Details*.(8,9)

*Proper procedures are given in *Backfilling Utility Trenches*.(7)

**For engineers familiar with the California bearing ratio method (CBR) or resistance values (r), a comparison is shown in Fig. 1.

Longitudinal Joints

Longitudinal joints are installed to control longitudinal cracking. They usually are spaced to coincide with lane markings–at 8- to 12-ft. intervals. Joint spacing should not be greater than 13 ft. unless local experience has shown that the pavements will perform satisfactorily. The depth of longitudinal joints should be at least one-fourth of the pavement thickness plus one-half inch.

Most integral curb city street pavements are restrained by the backfill behind the curbs, which eliminates the need for tying longitudinal joints with deformed tiebars or tiebolts.

Transverse Joints

Transverse contraction joints are used to control transverse cracking. Contraction joints relieve (1) tensile stresses that occur when the slab contracts and (2) curling stresses caused by differential temperatures and moisture contents within the slab. Most contraction joints are constructed by sawing after the concrete has set, by hand-forming, or by inserting a preformed material into the plastic concrete. Selection of the method to be used normally is based on the weather conditions prevailing during construction, the characteristics of the aggregate, and the economies of the operation. In any case, the depth of the joints in city streets should be equal to one-fourth of the pavement thickness.

Distributed steel or wire mesh, as normally used, only serves to hold the edges of cracks tightly together. Steel in amounts for this use does not add appreciably to the structural strength of the pavement. If transverse contraction joints are properly spaced, no intermediate cracking should occur and distributed steel should be omitted. Thus it is necessary to determine the contraction-joint spacing that will control cracking. Usually this is 15 to 20 ft. The best guide is the experience gained from streets in service.

The need for load transfer devices in transverse contraction joints depends on subgrade conditions and the service to be required of the pavement. **Dowel bars are not needed in residential pavements or other light traffic streets, but they may be needed on arterial streets designed to carry heavy volumes and weights of truck traffic.**[10]

When transverse contraction joints are properly spaced, expansion joints are not required except at fixed objects and unsymmetrical intersections, provided that–

1. The pavement is built of materials with normal expansion characteristics.
2. Contraction joints are spaced at short intervals that will prevent formation of intermediate cracks.
3. The pavement is built when the ambient temperature is well above freezing.

If pavement is built in cold weather or if abnormally expansive materials are used, expansion joints are necessary–spaced at 600- to 800-ft. intervals.

CONSTRUCTION SPECIFICATIONS

No matter how meticulously a structure is designed, it cannot serve its intended function unless it is built with every precaution to ensure quality workmanship in the final product. For this, adequate specifications are necessary. But the specifications are not enough unless they are accurately enforced by competent inspection. Suggested specifications for city street pavement are available from the Portland Cement Association.[11]

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2. Robbins, E. Guy, and Warnes, Cloyd E., "Traffic Data for Concrete Pavement Design," *Transportation Engineering Journal, Proceedings of the American Society of Civil Engineers*, TE1, Paper 9555, February 1973, pages 17-29.
3. *Thickness Design for Concrete Pavements*, Portland Cement Association, 1966.
4. *Scale-Resistant Concrete Pavements*, Portland Cement Association, 1964.
5. *Design and Control of Concrete Mixtures*, Portland Cement Association, 1968.
6. *Subgrades and Subbases for Concrete Pavements*, Portland Cement Association, 1971.
7. *Backfilling Utility Trenches*, Portland Cement Association, 1964.
8. *Integral Curb Pavement, Typical Sections and Details*, Portland Cement Association, 1965.
9. *Integral Curb Pavement, Typical Sections and Details–for Residential Streets*, Portland Cement Association, 1965.
10. *Joint Design for Concrete Pavements*, Portland Cement Association, 1961.
11. Suggested Specifications for Construction of Integral-Curb Concrete Streets, Portland Cement Association, 1965.

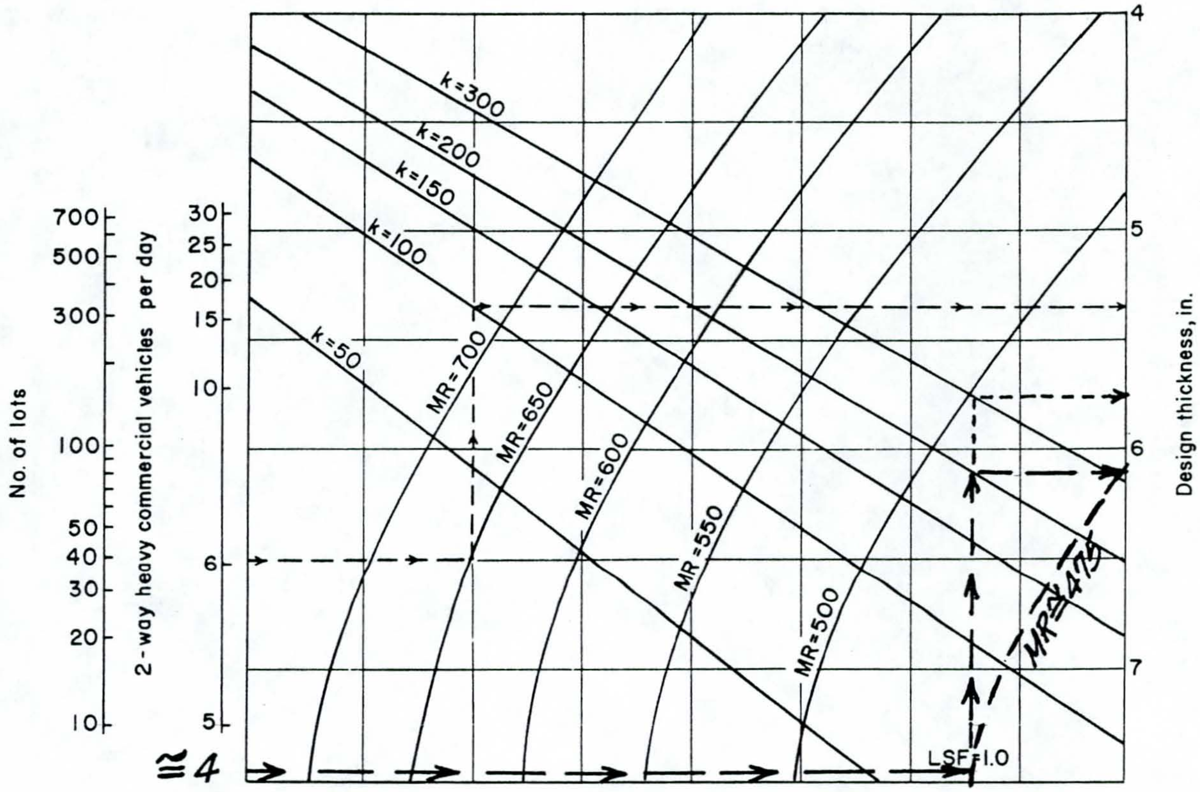


Chart 1. Thickness design chart for residential and residential collector streets for 35-year design life.

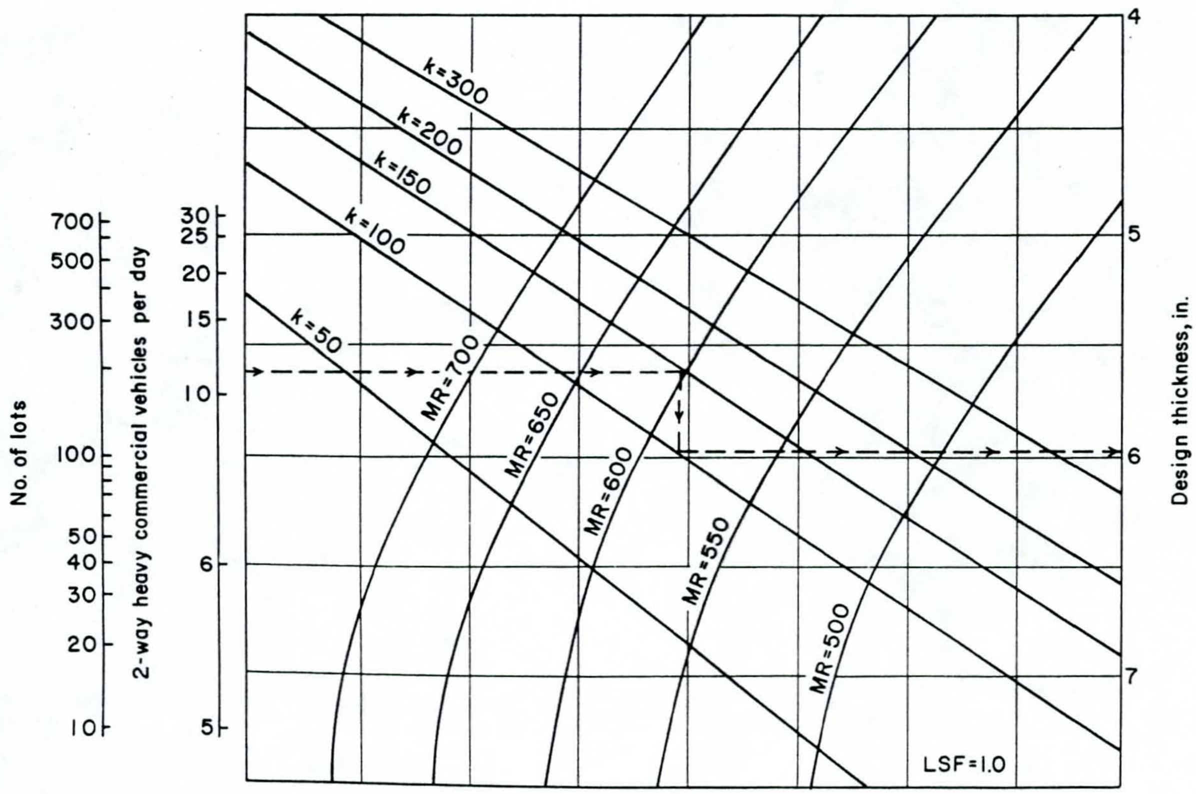


Chart 2. Thickness design chart for residential and residential collector streets for 50-year design life.

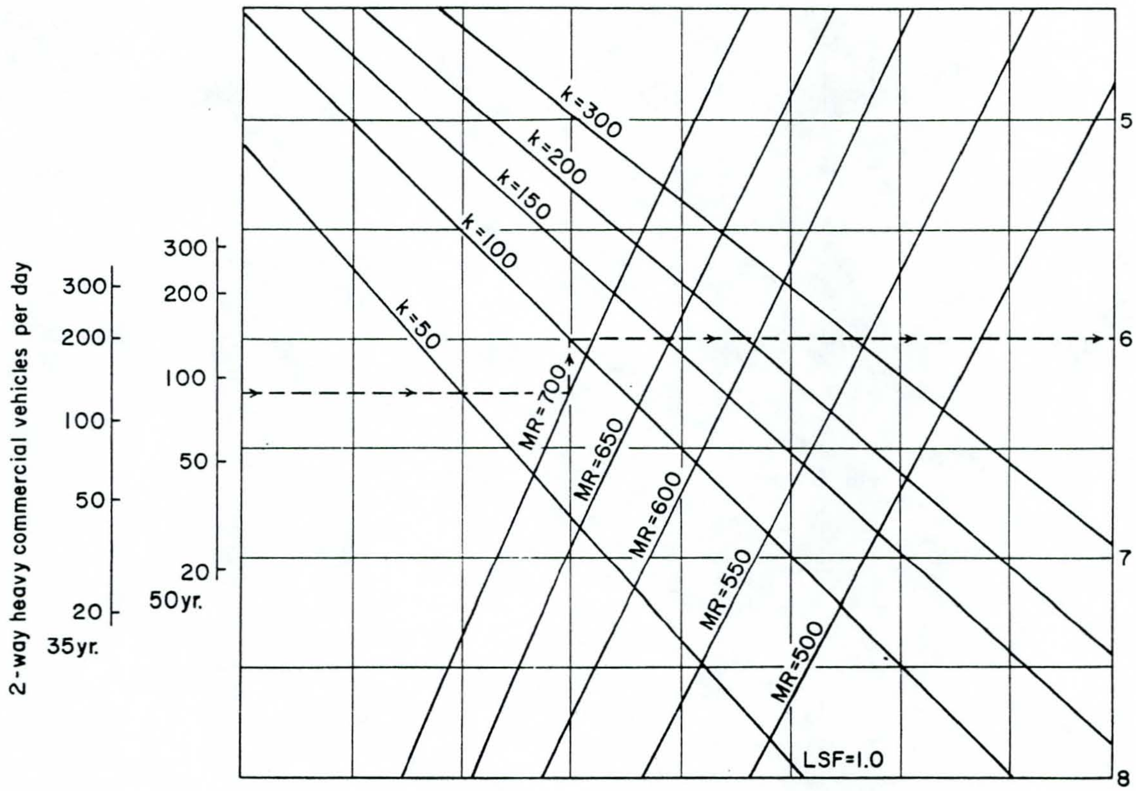


Chart 3. Thickness design chart for collector streets for 35- and 50-year design life.

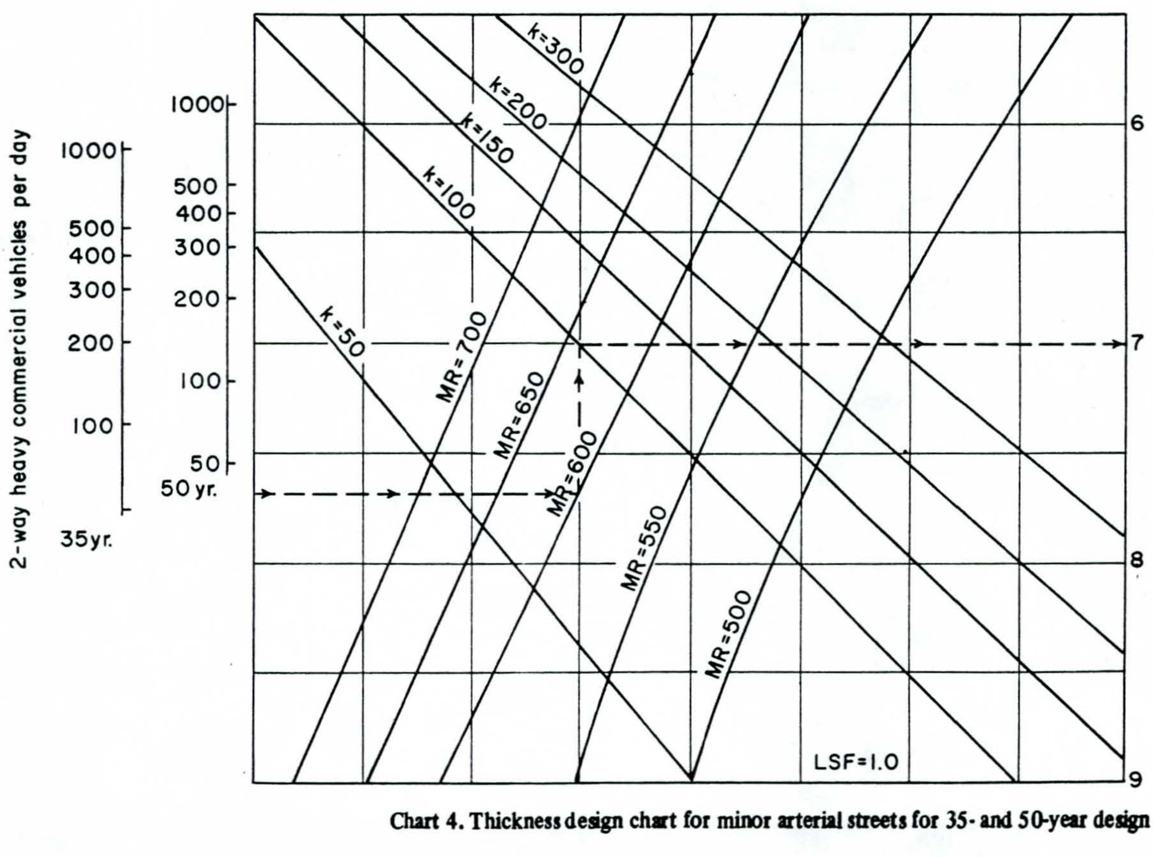


Chart 4. Thickness design chart for minor arterial streets for 35- and 50-year design life.

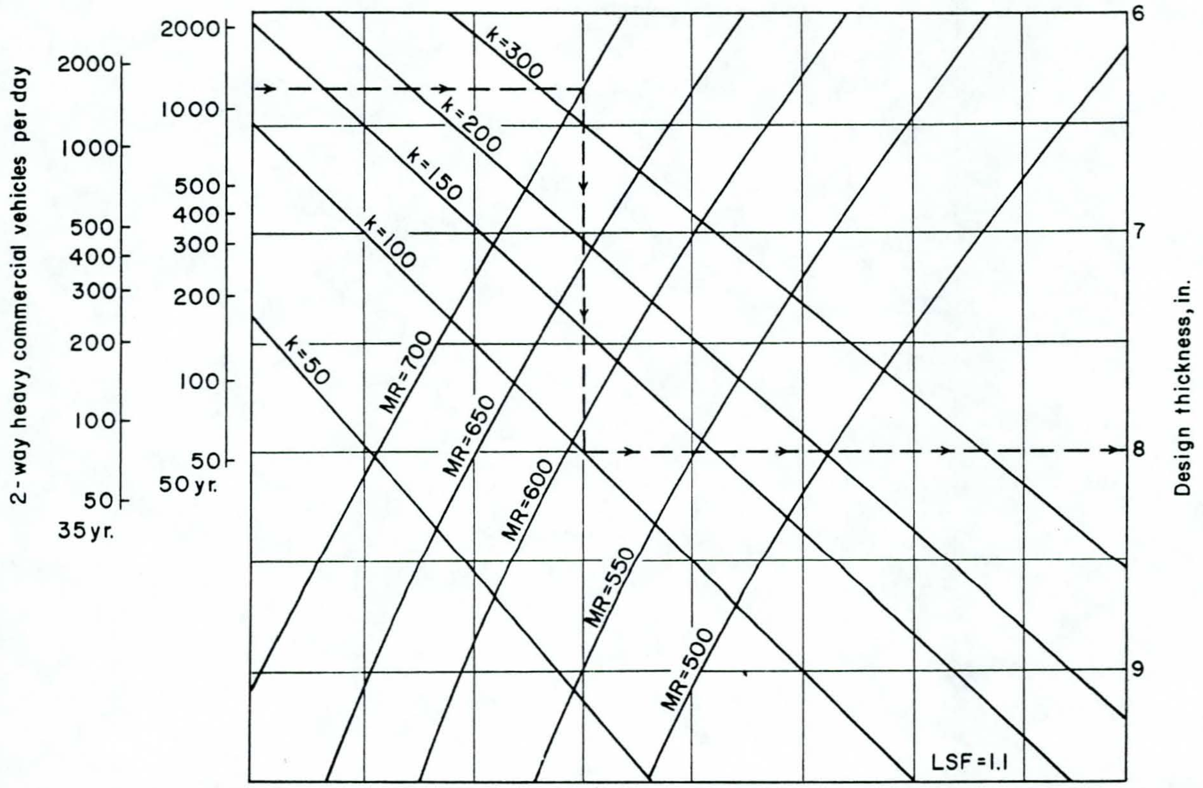


Chart 5. Thickness design chart for arterial and business streets for 35- and 50-year design life.

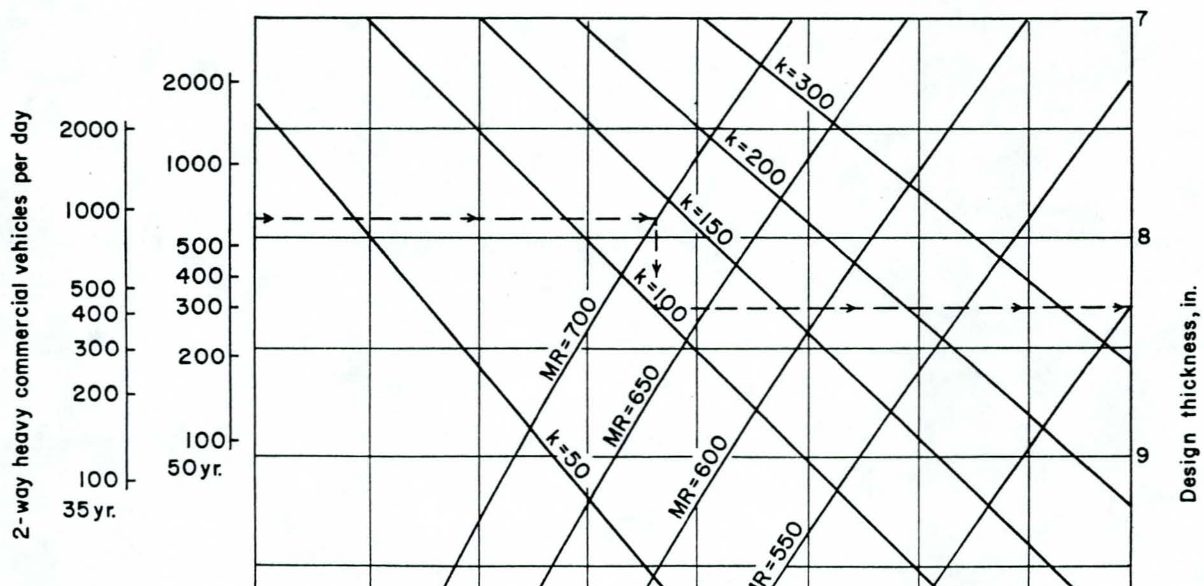
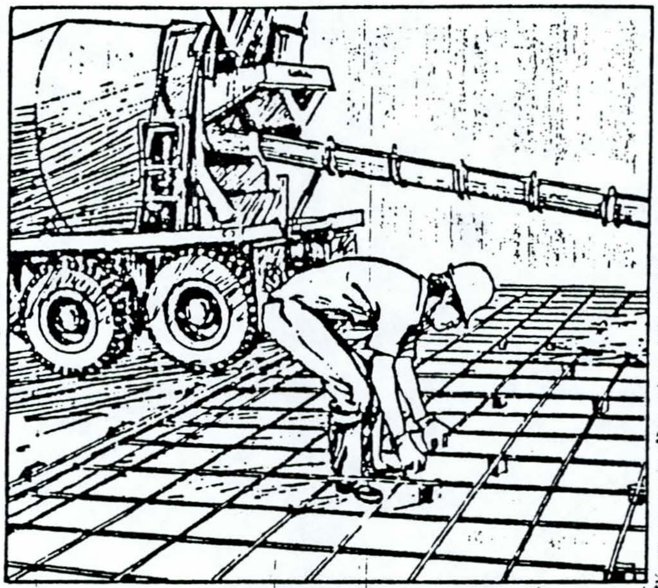


Chart 6. Thickness design chart for major arterial and industrial streets for 35- and 50-year design life.



DESIGN of INDUSTRIAL FLOORS

Publication SCM-5 (83)

Doweled Control Joints

In most industrial and commercial floors on ground, two mechanisms are used to help transfer moving loads across joints: keyways and dowels.

A keyed joint is made by attaching a beveled strip of wood, or a preformed key, to the bulkhead against which the slab is cast. The depression left in the edge of the slab after the bulkhead is stripped is coated with bondbreaker and then filled with concrete when the next slab is placed, thus, in theory, keying the slabs together. But in practice a keyed joint does not always remain tight. As the floor slabs shorten due to drying shrinkage, the key loses contact with its matching recess. Then as loads roll over the joint, the slab edges deflect. This loss of load transfer is an inherent weakness of keyed joints, especially in heavily loaded floors.

The other method of load transfer—steel dowels—seems to work better. The dowels continue to distribute the load after the slabs shrink and pull apart. As noted previously, control joints that open wider than 0.035 in. are less than 100% effective at transferring wheel loads across the joints in a concrete floor. However, dowels can be used to supplement the load transfer produced by aggregate interlock.

Placed at middepth in the slab, dowels resist shear as loads cross the joint and thus help to reduce deflections and stresses at the joint. Dowels should be smooth round steel bars conforming to ASTM A615. Recommended dowel dimensions and spacing are in Table 7.

Dowels should be free to move longitudinally in their slots to accommodate joint movements due to expansion or contraction of the concrete slab. Accordingly, before delivery to the jobsite, at least one-half of each dowel bar should be coated with a bondbreaking substance such as one coat of lead or tar paint. If this is not done, lubricants must be applied in the field to reduce friction. Plastic dowel sleeves may be used instead of coating.

Dowels in Reinforced Slabs

In floor slabs reinforced with distributed steel for crack control, the control joints are usually spaced too far

Fig. 19. Reinforced (mesh-dowel design) floor slab.

apart—more than 15 ft— to gain much benefit from aggregate interlock. Load transfer at these joints can be enhanced by using dowels (see Fig. 19).

The integrity and serviceability of intermediate random cracks that may occur between control joints is ensured by the distributed steel in reinforced slabs. But the distributed steel must be properly sized, located 2 in. below the slab surface, and discontinued at all control joints in order to keep intermediate cracks tightly closed and ensure aggregate interlock for effective load transfer at the cracks.

Since dowels are designed to slip, permitting slab movement at the joints, they must be accurately aligned parallel to the floor surface and the centerline of the slab.

REINFORCEMENT FOR FLOORS ON GROUND

Is reinforcement necessary?

NO
- WITH UNIFORM SUPPORT AND SHORT JOINT SPACING

YES
- WHEN LONG JOINT SPACING IS REQUIRED
- WHEN JOINTS ARE UNACCEPTABLE IN FLOOR USE

When a long joint spacing is selected to minimize the number of joints (shrinkage cracks may occur within the panel) or when joints are unacceptable in floor use, then distributed-steel reinforcement is placed in the slab.

The primary purpose of reinforcement in a slab on grade is to hold tightly closed any cracks that may occur between the joints so that aggregate interlock can function. It does not prevent cracking nor does it increase significantly the load-carrying capacity of the floor. Since critical flexural stresses occur in the top as well as the bottom of concrete floors, the steel would have to be placed in two layers to resist the stresses. To illustrate, if the relatively small amount of reinforcement normally used in floor slabs could be placed near both the top and bottom of the slab, in a 6-in. slab the load-carrying capacity would be increased by approximately 3%.

The amount of steel reinforcement needed can be determined by the traditional subgrade-drag method as in pavement design. This method provides sufficient steel area to resist the tensile stress carried by the steel across cracks that develop as a result of subgrade restraint to slab movement. The formula to determine the amount of steel per linear foot is

$$A_s = \frac{FLw}{2f_s}$$

in which

A_s = cross-sectional area of steel, in square inches per linear foot of slab.

F = coefficient of subgrade friction. (Designers use 1.5 or 2.0 for pavements; 1.5 is recommended for concrete floors on ground.)

L = slab length (or width if appropriate) between free ends, in feet. (A free end is any joint free to move in a horizontal plane.)

w = weight of slab, in pounds per square foot. (For regular weight concrete, designers use 12.5 pounds per inch of floor thickness.)

f_s = allowable working stress of reinforcement, in pounds per square inch. (The working stress of steel is usually 0.67 or 0.75 times the yield strength of the steel in pounds per square inch.)

The values shown in Fig. 20 were calculated by the subgrade-drag method. A value of 0.75 of the yield strength was used because the consequences of a reinforcement failure are much less important than in normal reinforced concrete structural work.

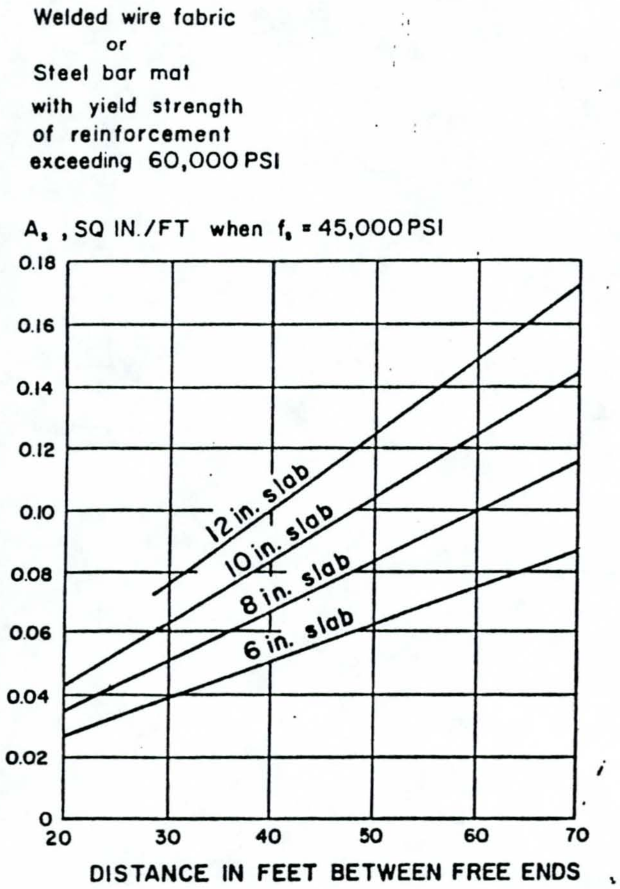


Fig. 20. Selection chart for distributed steel.

UNJOINTED FLOORS

An unjointed floor can be constructed when joints are unacceptable. Three methods are suggested:

1. A prestressed floor can be built through use of post-tensioning, a method in which steel strands are pulled taut after the concrete hardens to put a compressive stress in the concrete. This will counteract the development of tensile stresses and provide a crack-free surface. Large areas, 10,000 sq ft and more, can be constructed in this manner without joints.
2. Concrete made with expansive cement can be used to offset the amount of drying shrinkage to be anticipated after curing. Control joints are not needed when construction joints are used at intervals of 40 to 120 ft. Large areas, to 20,000 sq ft, have been cast in this manner without joints.
3. Large areas can be cast without control joints when the amount of distributed steel is about one-half of 1% of the cross-sectional area of the slab. Special effort should also be made to reduce subgrade friction in floors without control joints.

CONSTRUCTION

Workmanship

All parties—owner, designer, contractor—must agree prior to bidding on the level of quality or class of workmanship that will be necessary for the floor job they want. A good concrete floor on ground is the result of sensible planning, careful design and detailing, complete specifications, proper inspection, good workmanship, and the good intentions of each person sharing responsibility for the end result.

Subgrades

Good construction begins with a well-prepared subgrade. Many floor problems can be traced to poor subgrade preparation. A poorly compacted and prepared subgrade ranks high as a cause of settlement cracking and failure to carry the applied loads.

The subgrade should be uniform, firm, and free from all sod, grass, humus, and other rich organic matter as these materials will not compact to give good support to the floor. Subgrade support should be reasonably uniform with no abrupt changes from hard to soft spots within the floor area. To construct a reasonably uniform subgrade, special care must be taken to ensure that there is control of the three major causes of nonuniform support: (1) expansive soils, (2) hard spots and soft spots, and (3) backfilling.

The subgrade must be brought to within required tolerances at the specified grade and level. The use of laser alignment tools to control grading operations will make the surface as level as possible; or a scratch templet can be used to reveal high and low spots. Occasionally a choker fill should be added to bind the surface before final compaction with a vibratory roller, heavy-plate vibrator, or a tandem roller. A reasonably accurate, level subgrade will ensure that the correct thickness of concrete is placed. If the surface is too uneven, concrete will be wasted and the potential for random cracking will be increased. The subgrade should be moist when concrete placement begins (Fig. 21).

5. COMMON STOCK STYLES OF WELDED WIRE FABRIC

Certain styles of welded wire fabric as shown in Table I have been recommended by the Wire Reinforcement Institute as common stock styles. Use of these styles is normally based on empirical practice and quick availability rather than on specific steel area designs. Styles of fabric produced to meet other specific steel area requirements are ordered for designated projects, or, in some localities, may be available from inventory. For comparative purposes, the previously used style designations by steel wire gage are included in Table I.

TABLE I — COMMON STOCK STYLES OF WELDED WIRE FABRIC

STYLE DESIGNATION		Steel Area Sq. In. Per Ft.		Weight Approx. Lbs. Per 100 Sq. Ft.
New Designation (By W-number)	Old Designation (By Steel Wire Gage)	Longit.	Transv.	
ROLLS				
6x6—W1.4xW1.4	6x6—10x10	.03	.03	21
6x6—W2xW2	6x6—8x8*	.04	.04	29
6x6—W2.9xW2.9	6x6—6x6	.06	.06	42
6x6—W4xW4	6x6—4x4	.08	.08	58
4x4—W1.4xW1.4	4x4—10x10	.04	.04	31
4x4—W2xW2	4x4—8x8*	.06	.06	43
4x4—W2.9xW2.9	4x4—6x6	.09	.09	62
4x4—W4xW4	4x4—4x4	.12	.12	86
SHEETS				
6x6—W2.9xW2.9	6x6—6x6	.06	.06	42
6x6—W4xW4	6x6—4x4	.08	.08	58
6x6—W5.5xW5.5	6x6—2x2**	.11	.11	80
4x4—W4xW4	4x4—4x4	.12	.12	86

*Exact W-number size for 8-gage is W2.1.
**Exact W-number size for 2-gage is W5.4.

From Manual -
Concrete Reinforcing Steel Institute

The safety factor can be selected by either of two ways. When the full-rated capacity of the forklift truck is selected to determine slab thickness, a safety factor between 1.5 to 1.7 is suggested because forklift trucks are not always operated at full-load capacity. When realistic-load data are known and used, a safety factor of 2 is suggested.

Shrinkage Stress

Shrinkage stresses are not considered to be significant. For example, a shrinkage stress of 23 psi is computed for an 8-in. slab jointed at 20 ft using a subgrade friction factor of 1.5. Pavement research has shown, however, that the actual stress developed will be much less—only one-third or one-half of that predicted.

Impact

Some procedures for pavement design increase the wheel loads by a factor to accommodate the effect of wheel impact. A load-impact factor is not included in the PCA floor design procedure (except in the safety factor) because this procedure is based on pavement research that shows slab stresses are less for moving loads than for static loads.

Flexural Stress

The flexural stresses indicated on the design charts are computed at the interior of a slab. When the slab edges at all joints have adequate load transfer (by means of dowels, keyways, or aggregate interlock), it is assumed that the panel area acts as a portion of a continuous large-area slab.

At free edges that lack adequate load transfer, concentrated loads will produce stresses that are somewhat greater than those for the interior. Because of this, if lift trucks will pass over an isolation joint (at a doorway for instance) the slab should be thickened by 25% gradually over a distance of 5 ft. Thickened sections (edges) restrain horizontal movement that may cause cracking in the interior of the slab. Mechanical devices (dowels) may be preferable for load transfer.

The assumption of interior load placement, combined with the choice of an appropriate safety factor and adequate concrete strength, gives a reasonable basis for floor thickness design.

Preliminary Design

For preliminary design purposes, or when detailed design data are not available, Fig. 3 can be used to select slab thickness based on the rated capacity of the heaviest lift trucks that will operate on the floor. The chart was prepared for typical lift trucks from manufacturers' data shown in Table 4. It cannot be used for trucks with capacity and wheel-spacing data that differ substantially from the data in Table 4.

The combination of a low k value and a low working stress in Fig. 3 results in a conservative slab thickness.

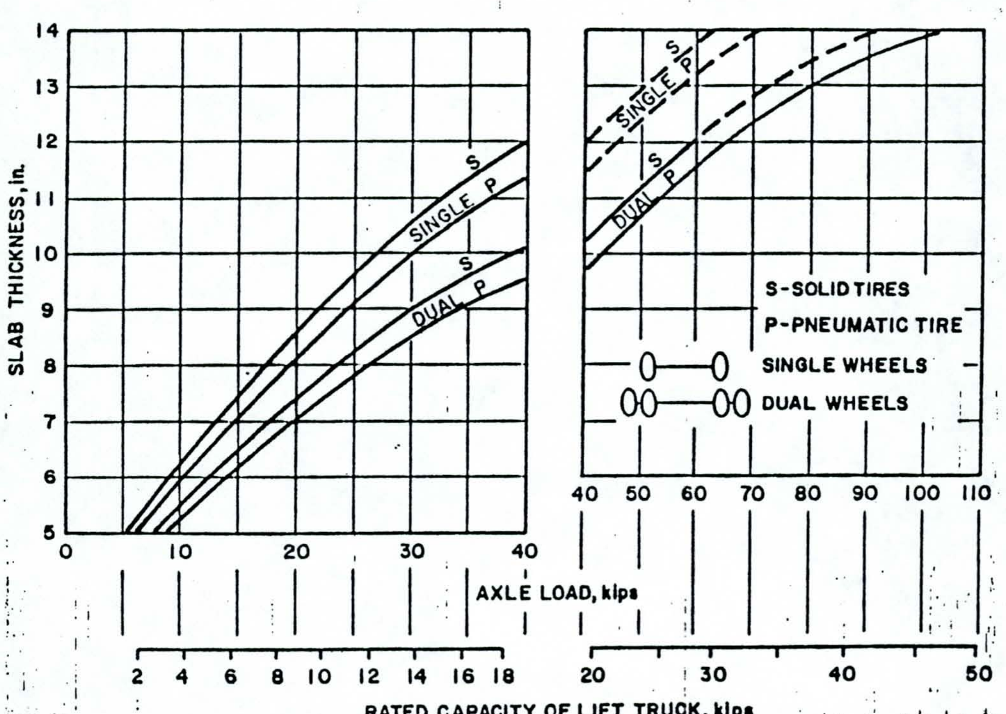


Fig. 3. Estimated slab thicknesses for lift trucks (based on average truck data shown in Table 4 and conservative design assumptions of k = 50 pci, concrete working stress = 250 psi).