Appendix G: Hydrology Study

FINAL
Hydrologic Study
for the
Walters Road West Project

Prepared for:

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SECTION 1: INTRODUCTION

1.1 - Purpose and Background

Michael Brandman Associates (MBA) has prepared this Hydrologic Study (Study) to analyze the offsite drainage impacts of the Walters Road West Commercial Project (Project). This Study provides supplemental technical information and data necessary to support the discussions and findings in the environmental document for the Project. This Study also provides an evaluation of potential water quality effects as a consequence of the Project.

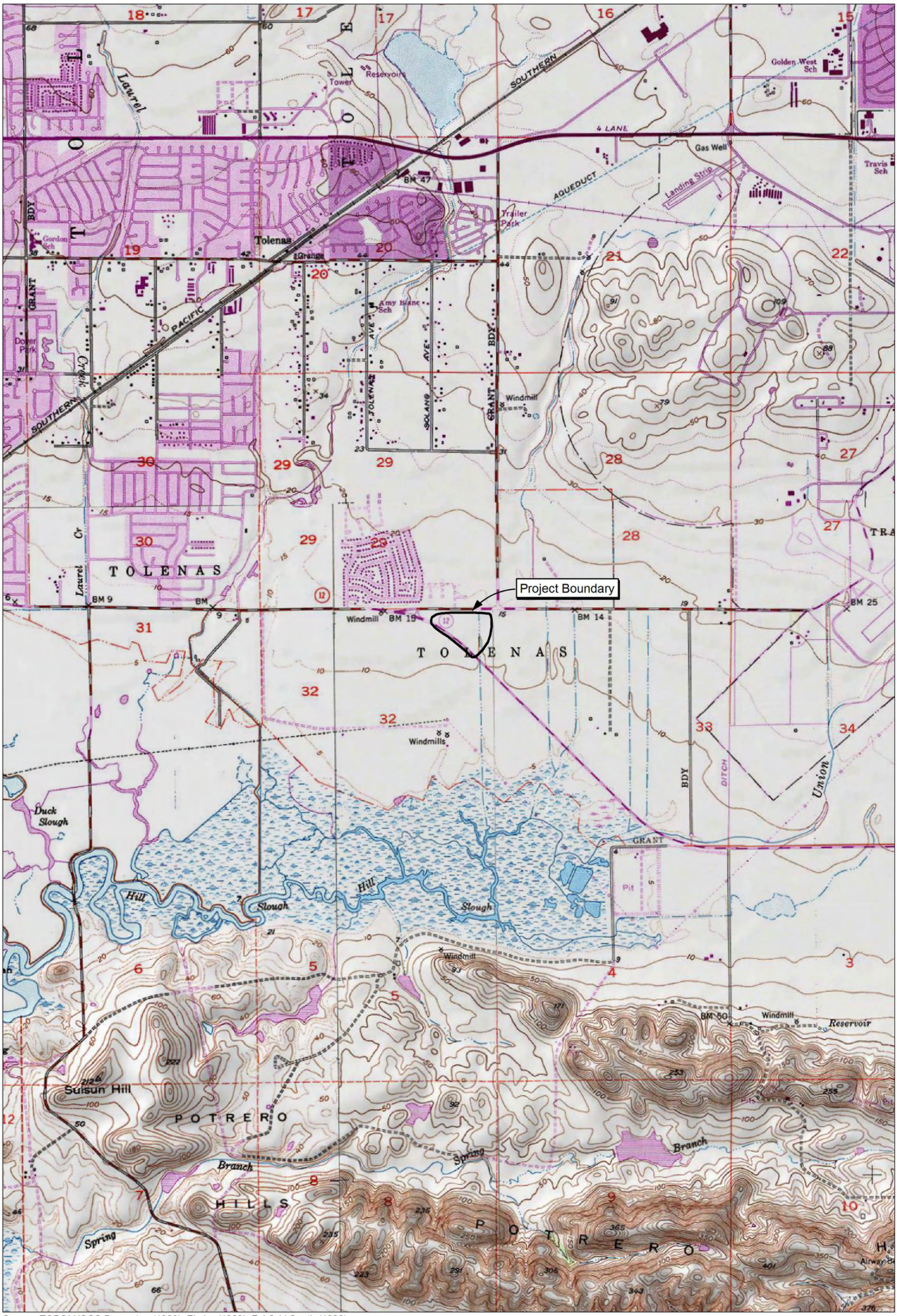
The following sections focus on the hydrologic conditions that characterize the City of Suisun (City), the Project locale, and offsite hydrologic factors that influence localized drainage and surface water quality. This Study includes storm runoff estimates derived through use of the rational method, based on guidance provided in the City's Engineering Standards. The model output and the hydrologic conditions evaluated in this Study provide the City with the basis for assessing drainage impacts of the Project on existing stormwater runoff rates, existing drainage infrastructure, and how those changes could affect water quality.

1.2 - Project Location

The Project area is located in south-eastern Suisun City, in central Solano County, California. The 20.8-acre Project site is situated on an undeveloped lot immediately north of the intersection of Walters Road and State Route (SR) 12. As depicted on the Denverton California, United States Geological Survey (USGS) 7.5 minute topographic quadrangle, the Project site is located in an unsectioned area of Township 4 North, Range 1 West (USGS Dataviewer, Modified 09/14/2005) (see Exhibit 1). As shown in Exhibit 2 (Aerial), the Quail Glen subdivision is adjacent and to the north of the Project site. SR 12 abuts the southern perimeter of the Project site with the Lawler Ranch subdivision located adjacent further south. The northern end of the Suisun Marsh is located immediately to the south of Lawler Ranch. This location roughly corresponds with the U.S. National Grid (USNG) Coordinates 10S EH 89630 32325 (NAD83) and Universal Transverse Mercator (UTM) Coordinates 10 589630 4232325.

1.3 - Project Description and Stormwater Controls

The Project consists of constructing a 214,919 square foot (sf) Wal-Mart Supercenter on 18.34 acres of the 20.8-acre site. The Wal-Mart will also contain a 24,653 sf garden center. Along the front of the Supercenter will be a series of small internal shops occupying a total floor area of 7,007 sf. A 4,100 sf full-service gas station and related uses retail building will be constructed on 1.05-acres. The remaining 1.40-acres of developable space will contain an 8,000 sf building intended for a sit-down restaurant.



Source: TOPO! USGS Denverton (1980), Elmira (1980), Fairfield South (1980), Fairfield North (1980) 7.5' DRG; and MBA GIS (2007).

NORTH

2,000 1,000 0 2,000 Feet

Michael Brandman Associates

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Exhibit 1
Local Vicinity Map
Topographic Base

Source: USDA NAIP Solano (2005), City of Susuin City (2007), and MBA GIS (2007).

NORTH

2,000 1,000 0 2,000 Feet

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Exhibit 2
Local Vicinity Map
Aerial Base

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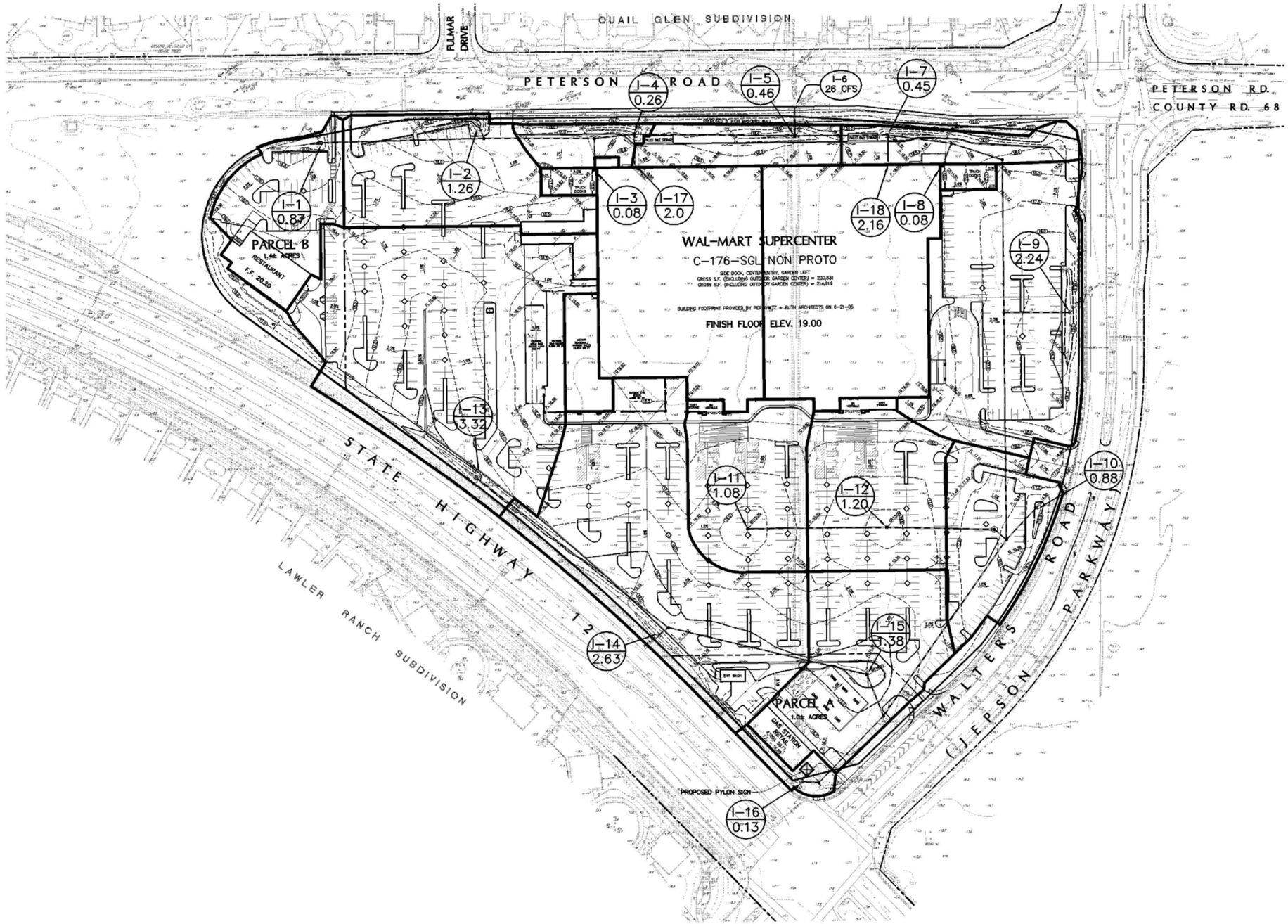
The Project Stormwater Control Plan (SCP) identifies the site impervious areas which have been divided into eleven drainage management areas (DMAs) notated by dark dashed lines in Exhibit 3 (SCP)(Robert A. Karn and Associates, 2007). These eleven DMAs consist of five paved and six roof areas as shown in Exhibit 3. Each DMA is labeled as PAVE-xx or ROOF-xx for areas of pavement or roof and include the approximate size of the area in square feet. Pervious self-retaining areas within the DMAs are shaded and are labeled as SR-xx and include the approximate size of the area in square feet (see Exhibit 3). Runoff from each of these impervious DMAs is managed by routing to a vegetated swale, infiltration planter, or proprietary device sized to treat runoff from that area (Robert A. Karn and Associates, 2007).

1.4 - City Drainage Standards and Specifications

Chapter 4.0 of the City's Engineering Standards and Specifications (Standards) provides general requirements for the handling of stormwater runoff from new development projects. These standards are intended to implement surface water regulations at the local level and that runoff from storms up to the 100-year return frequency are conveyed through storm facilities and disposed of in a manner that protects public and private improvements from flood hazards. The final Project Stormwater Plan (10/9/2006) prepared by RAK Civil Engineering for the Project, will be required to satisfy the requirements outlined below.

> The Standards require that <u>all</u> proposed storm drainage facilities include provisions for future upstream development and no development shall discharge at a rate which exceeds the capacity of any portion of the existing downstream system. Consistent with this requirement, calculations for storm drainage design within a development as well as calculations for runoff generated by upstream areas within the contributing watershed are provided in this report.
>
> The City requires the containment of flood waters within the public right-of-way (e.g. streets or other approved right-of-ways) at all times through grading, levees or alternative means acceptable to the City Engineer. The City requires that in no instance shall an improvement be designed such that flood waters reach a depth of 0.50 feet, as measured from the top-of-curb, before overland release occurs. The City does not permit overland releases between lots.
>
> The City's Standards require that in instances where drainage conveyance pipelines intersect, the downstream pipe is required to have a crown elevation, which is less than or equal to, the crowns of all upstream connecting pipes. In addition, the City requires that the diameter of drainage conveyance pipeline not decrease in the downstream direction.



Source: RAK Civil Engineers, 2006.

NORTH

174 87 0 174

SCALE IN FEET

Michael Brandman Associates

Exhibit 3
Project Stormwater Control Plan

The City requires that storm drain systems be designed to prevent contamination of creeks and streams with polluted or silt-laden storm drainage. The design of proposed drainage improvements should include best management practices (BMPs) that comply with the standards for the National Pollutant Discharge Elimination System (NPDES) as stipulated by the U. S. Environmental Protection Agency (USEPA) and the State Water Quality Control Board (SWRCB).

1.5 - Stormwater Regulations

The City of Suisun City is currently subject to National Pollutant Discharge Elimination System (NPDES) Permit No. CAS612005 issued under Order No. 95-079 on April 19, 1995 and amended through Order No. R2-2003-0034. Provision C.3 of the Permit, which identifies new development and redevelopment performance goals, is intended to address pollutant discharges and changes in runoff flows through implementation of post-construction treatment measures, source control, and site design measures, to the maximum extent practicable.

The Project includes the construction of over one acre (43,560 square feet) of impervious surface area and, therefore, is subject to Provision C.3 of the Order. Provision C.3 requires the incorporation of appropriate source control and site design measures to minimize stormwater pollutant discharges to the maximum extent practicable. The City as a co-permittee is required to include conditions of approval in permits for subject to Provision C.3.c to ensure that stormwater pollutant discharges are reduced by incorporation of treatment measures and other appropriate source control and site design measures, and increases in runoff flows are managed in accordance with C.3.f, to the maximum extent practicable. The conditions are required, to at minimum, address the following goals:

(i) Require a project proponent to implement site design/landscape characteristics where feasible which maximize infiltration (where appropriate), provide retention or detention, slow runoff, and minimize impervious land coverage, so that post-development pollutant loads from a site have been reduced to the maximum extent practicable; and

(ii) For new and redevelopment projects that discharge directly (not mixed with runoff from other developed sites) to water bodies listed as impaired by a pollutant(s) pursuant to CWA Section 303(d), ensure that post-project runoff does not exceed pre-project levels for such pollutant(s), through implementation of the control measures addressed in this provision, to the maximum extent practicable, in conformance with Provision C.1.

SECTION 2: DESCRIPTION OF LOCALIZED WATERSHED CONDITIONS

2.1 - Existing Conditions

2.1.1 - Current Land Use

The Project area is generally urbanized with various forms of residential and commercial development to the north, south, and west of the Project site (see Exhibit 2). With the exception of undeveloped areas to the east, the Project site is the only undeveloped lot within the immediate Project area. However, lands to the east are planned for future development and are identified as urban reserve and light industrial in the City's General Plan (Suisun City, 1992). In addition, Travis Air Force Base is located further east at the terminus of Peterson Road. The 27,500-acre Suisun Marsh is located to the south of the Lawler Ranch subdivision and is considered a significant open space resource.

The Project site is planned for urban development and is designated for General Commercial (GC) under the City's General Plan. Currently, the Project site is comprised of rudural vegetation and is generally level; slightly grading to the south. Evidence of disking within the last two years is evident across the site. A linear, drainage ditch, approximately 1,025 feet in length, vertically bisects the site in a north-south orientation. Much of the ditch contains emerging vegetation suggesting a lack of regular channel maintenance. However, the southern end of the ditch exhibits evidence of recent channel excavation with ponding evident at the time of MBA's March 1, 2007 site reconnaissance.

2.1.2 - Soils and Geomorphology

The project site is in the northern Coast Ranges Physiographic Region in an area consisting of flat-lying stream-laid deposits (alluvium) overlying Pliocene-aged (2 to 5.3 million years old) bedrock composed of sedimentary rocks derived from mixed sources (CGS, 2002). Northern portions of the Project area are mapped as being underlain by Pleistocene-aged (< 1.6 million years ago) alluvium (Qpa). Areas to the south of SR 12, including a small southern section of the Project site, are mapped as being underlain by Holocene-aged (<10,000 years ago) alluvium (Qha). Further south, the underlying deposits are mapped as Bay Mud (Qhym). (USGS, 2007).

The topography in the vicinity of the Project is characterized by south sloping alluvial plains interspersed with small valleys that extend into the tidal marshes north of Suisun Bay. Clement Hill, part of the Vaca Mountains, and Potrero Hill, part of the Montezuma Hills, to the south are the most prominent topographical features in the Project area rising up to elevations of 340 feet and 250 feet above mean sea level (msl), respectively. Elevations in the vicinity of the immediate Project area range from sea level along the Hill Slough to 20 feet above msl near the intersection of Walters Road and Montebello Drive to the north of the Project site. Onsite topography is generally level with existing site elevations ranging from 18 feet above mean sea level (msl) in the north-central portion of the site to 12 feet msl near the southern corner (RKA Civil Engineers, 2006).

The NRCS Soil Survey for Solano County, California (1977) describes surface soils across the site as Antioch-San Ysidro complex, 0 to 2 percent slopes (AoA), Antioch-San Ysidro complex, thick surface, 0 to 2 percent slopes (AsA), and Pescadero clay loam. These soils are generally derived from alluvial and marine sediments and have deep profiles (> 60 inches), low soil strength, and contain large fractions of clay at depth. The Fairfield-Suisun Urban Runoff Management Program (FSURMP) maps these soils as hydrologic soil group[1] "D" in recognition of these general characteristics and poor drainage conditions.

Antioch soils have a light brownish to brown, loam-textured surface horizon that transitions to a moderately alkaline clay or clay loam at depth. With increasing depth Antioch soils become more alkaline and, at depths below 20 inches, the exchangeable sodium increases. San Ysidro soils are very similar to Antioch soils in physical composition, but have a lower percentage of exchangeable sodium at depth. Pescadero soils are formed in basins and concave sloped areas and occur along the southern margin of the City. Pescadero soils typically contain a higher percentage of clay near the surface as opposed to the Antioch and San Ysidro soils. In addition, Pescadero soils are characterized by mottling below depths of 20 inches. Mottling is often associated with anoxic soil conditions, which is indicative of wetland soil hydrology.

2.1.3 - Climate

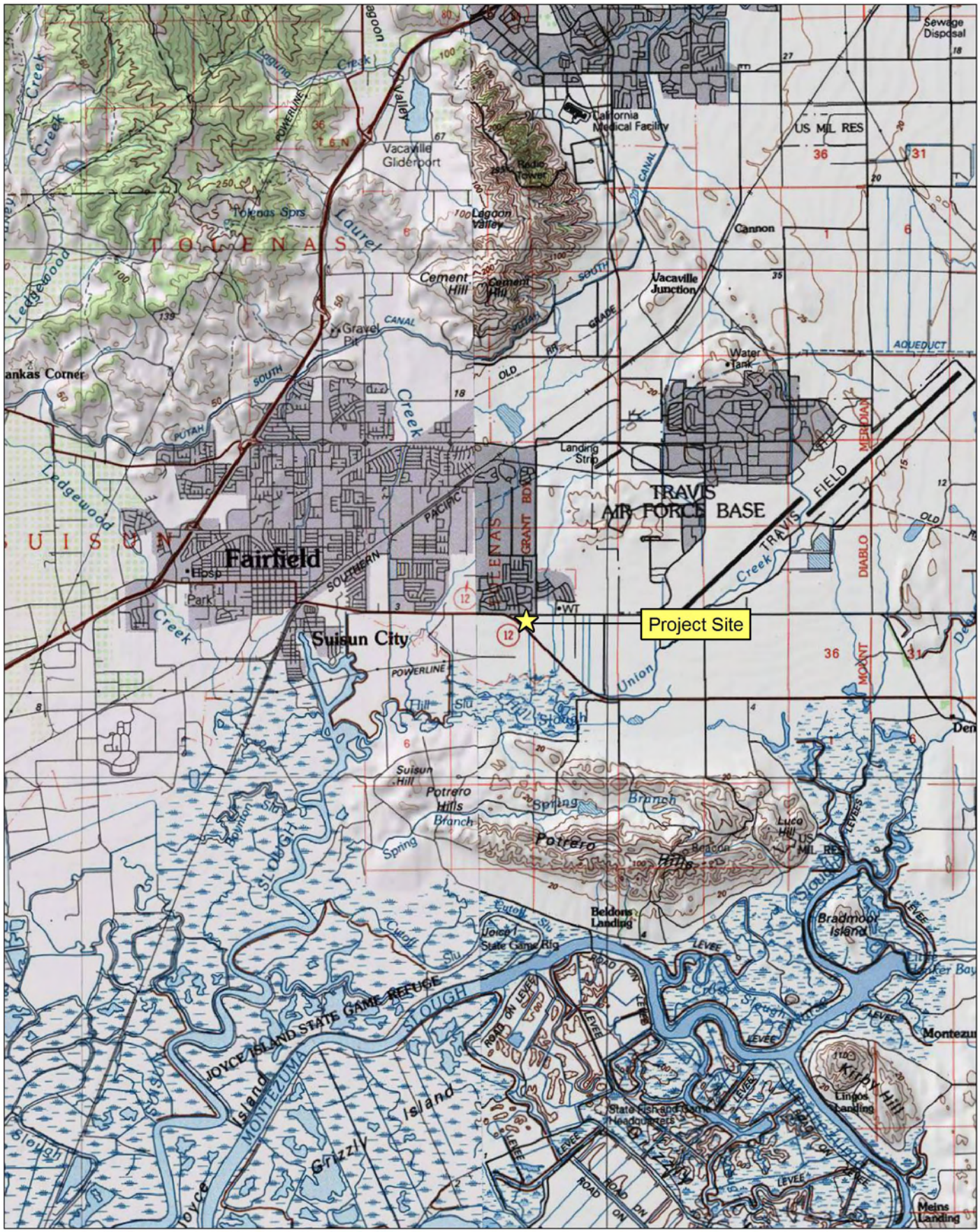
Precipitation in the Solano County is derived from frontal low-pressures systems that originate over the Pacific Ocean and travel generally west into California (SCWA Hydrology, 1999). The Project site is situated in the central Coast Range, which is characterized by cool, wet winters and dry, warm summers. The mean annual precipitation is 20 inches. The majority of the annual precipitation falls as rain during the period of November through April. The 10-year, 24-hour estimated maximum precipitation amount is 3.0 inches and the 100-year, 24-hour maximum precipitation amount is 4.5 inches for the Project area (Western Regional Climate Center, 1973).

2.1.4 - Existing Surface Water Hydrology

Regional Hydrology

Suisun City is situated on the north and east banks of Suisun Slough, which connects with Grizzly Bay and links Suisun City to the Sacramento River and the San Francisco Bay (see Exhibit 4). Grizzly Bay is a northern subembayment of Suisun Bay, which is composed of three main channels that flow east to west towards the Carquinez Strait. The deepest channel flows through Suisun Cutoff, north of Ryer Island, and along the southern end of Grizzly Bay. All three channels join at the Carquinez Strait where they continue through to the southern section of San Pablo Bay, and into San Francisco Bay.

[1] The hydrologic soil group is an identifier given to a soil which describes its ability to infiltrate water and produce water runoff. For example, a hydrologic soil group of A means that soil infiltrates water quickly, thus not producing much runoff, while a hydrologic soil group of D means that a soil infiltrates water slowly, thus producing a lot of runoff.



Source: TOPO! National Geographic Reference Map, California.

NORTH

1 0.5 0 1 Miles

Michael Brandman Associates

Exhibit 4
Regional Hydrology

The Department of Water Resources (DWR) maps the Project area within the Suisun Slough hydrologic subarea of the Fairfield hydrologic area. The Fairfield hydrologic area is located within the larger Suisun hydrologic unit (USGS Cataloging Unit: 18050001) (Calwater 2.2, 2006). The Carquinez Strait is located at the western portion of the Suisun hydrologic unit and the larger Sacramento-San Joaquin River Delta (Delta). The Delta's tributary areas include the Sacramento River, the Central Sierra, and the San Joaquin River basins accounting for approximately 41,300 square miles (USGS, 2004). The Delta is a triangular area of approximately 1,150 square miles, extending from Chipps Island near the City of Pittsburg on the west to Sacramento on the north and to Vernalis on the south. Much of the Delta is influenced by tidal action.

The Project area drains directly into Hill Slough, which empties into Suisun Slough, to the south-southeast into Grizzly Bay. Suisun Slough is fed by several local creeks including Laurel and Ledgewood Creek(s) to the west, Union Creek to the east, and locally McCoy Creek. McCoy Creek intercepts drainage runoff from much of northern and central portions of Suisun City and discharges to Hill Slough west of the Project site.

Local Surface Drainage

Drainage patterns within the Project vicinity are highly modified by existing development with runoff conveyed through a combination of engineered curb and gutter systems, roadside ditches, and linear, open channels. Open channels only remain on undeveloped lots such as the Project site and undeveloped, urban reserve properties to the east. These channels were observed to have substantial vegetative growth, which included, cattails, willows (*Salix spp.*), and Himalayan blackberry (*Rubis tricolor*). The Project site is located within a small, isolated drainage catchment located east of McCoy Creek. This drainage catchment is the primary drainage area under consideration in this report and is depicted in Exhibit 5.

As depicted in Exhibit 5, the upstream contributing drainage area is relatively small; only approximately 14.7-acres, with runoff originating from Peterson Road and a southeast section of Quail Glen subdivision. The 20.8-acre Project site represents over a forth of the $\pm$ 72-acre drainage area and, with the exception of the Hill Slough shoreline and adjacent Caltrans property, is the only undeveloped lot. In addition, the Project site contains one of the two linear drainage channels within the Project drainage area. This onsite drainage feature is responsible for conveying runoff across the Project site as illustrated in Exhibit 6. In addition, the southern end of the on-site channel is excavated and allows for ponding to occur, thereby providing a minor flow attenuation effect until the water level reaches the culvert, thereby allowing for off-site discharge (see Exhibit 7). The attenuation effect of this excavated area, based on the linear orientation of the ponded area, is estimated on the order of less than 4,500 cubic feet.

Source: USDA NAIP Solano (2005), City of Susuin City (2007), and MBA GIS (2007).

NORTH

1,000 500 0 1,000 Feet

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Exhibit 5
Project Drainage Area

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Photograph A - View of onsite drainage, from the south

Photograph B - View of the onsite drainage channel, from the north

Source: Michael Brandman Associates, 2007.

Michael Brandman Associates

Exhibit 6
Onsite Drainage Feature

Photograph A - northern extent of ponding

Photograph B - Ponding below the existing 36-inch culvert

Source: Michael Brandman Associates, 2007.

Michael Brandman Associates

Exhibit 7
Ponding in Onsite Channel

The other un-piped, drainage feature within the Project area is a roadside channel located south and parallel to the east bound lane of SR 12 (see Exhibit 8). These features along with runoff from portions of Walters Road converge south of the Project site and enter a 42-inch drainage line that extends south along Lawler Ranch Parkway. This line transitions to a 52-inch trunk line, concrete-cast pipe that discharges into Hill Slough via a submerged outfall approximately 150 feet beyond the southern perimeter of the Lawler Ranch subdivision. A reconnaissance of the storm drain outfall revealed that the structure is deteriorating and possibly in need of rehabilitation. The existing outfall is not equipped with a tidal flap gate, is cracked, and a sink hole is forming above a portion of the trunk line approximately 30 feet north of the outfall.

Normal tidal action has likely resulted in saltwater and bay sediments partially filling the trunk line upstream an undetermined distance. These sediments likely accumulate during the non-rainy season but are likely flushed out to some extent during high flows from the first significant storm event. As a result, outfall capacity and overall performance of the storm drain system in the area may be substantially reduced from its original design, potentially creating conditions that could lead to localized flooding during normal (e.g. 2-year interval) storm events. Storm drain clogging from sediment, trash, and other organic debris was observed within numerous drainage facilities including the rectangular culvert that bisects Peterson Road and discharges onto the Project site (Exhibit 9).

Flooding

The majority of the Project area is generally located on an alluvial fan and outside of a designated floodplain. The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map for the City of Suisun California, Solano County (FEMA, 2007) indicates that the outfall location is within a special shoreline flood hazard area and is the only area mapped as being inundated by the 100-year flood.

Tidal Influence

Hill Slough is under tidal influence. A tidal gate is located to the south of the Lawler Ranch Subdivision and approximately 400 feet southeast of the 52-inch outfall. Because of the local tidal influence, the 52-inch outfall is generally submerged and only exposed during low tide. This tidal influence creates a backwater effect within the local drainage system and decreases the hydraulic head in up-gradient areas. Based on data acquired for the Suisun Slough Entrance of Grizzly Bay, during the period of record—September 1977 through April 1979—the highest observed water level was 6.81 feet mean sea level (msl) on February 5, 1978 (Control Tide Station: 9415144 Port Chicago). The mean tide level was 2.45 feet msl and the lowest observed water level was -0.83 feet msl on December 12, 1977.

For reference purposes, the Lawler Ranch drainage outfall is designed based on a 3.0 feet msl tide elevation (Lawler Ranch Drainage Infrastructure Map, 1989).

View of SR 12 Roadside Channel, looking southeast

Source: Michael Brandman Associates, 2007.

Michael Brandman Associates

Exhibit 8
SR 12 Roadside Drainage

Sediment accumulating within the Box Culvert beneath Petersen Road

Source: Michael Brandman Associates, 2007.

Michael Brandman Associates

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Exhibit 9
Storm Drain Clogging

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Close-up of the submerged 52-inch outfall

Source: Michael Brandman Associates, 2007.

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Exhibit 10
Submerged Outfall

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HYDROLOGIC & HYDRAULIC ANALYSIS

SECTION 3: HYDROLOGICAL EVALUATION

3.1 - Methodology

Based on direction provided in Section 4.02 of the City's Standards, the Rational Method is recommended for use in estimating drainage discharges for a design storm event. The Rational Method is appropriate in situations where the drainage area is less than 640 acres and, therefore, is appropriate for the roughly 72- acre drainage area containing the Project site. The basic assumptions for the Rational Method are:

1. The maximum runoff rate at any design point is a function of the average rate of rainfall during the time of concentration.
2. The maximum rate of rainfall occurs during the time of concentration. The variability of the storm pattern is neglected.

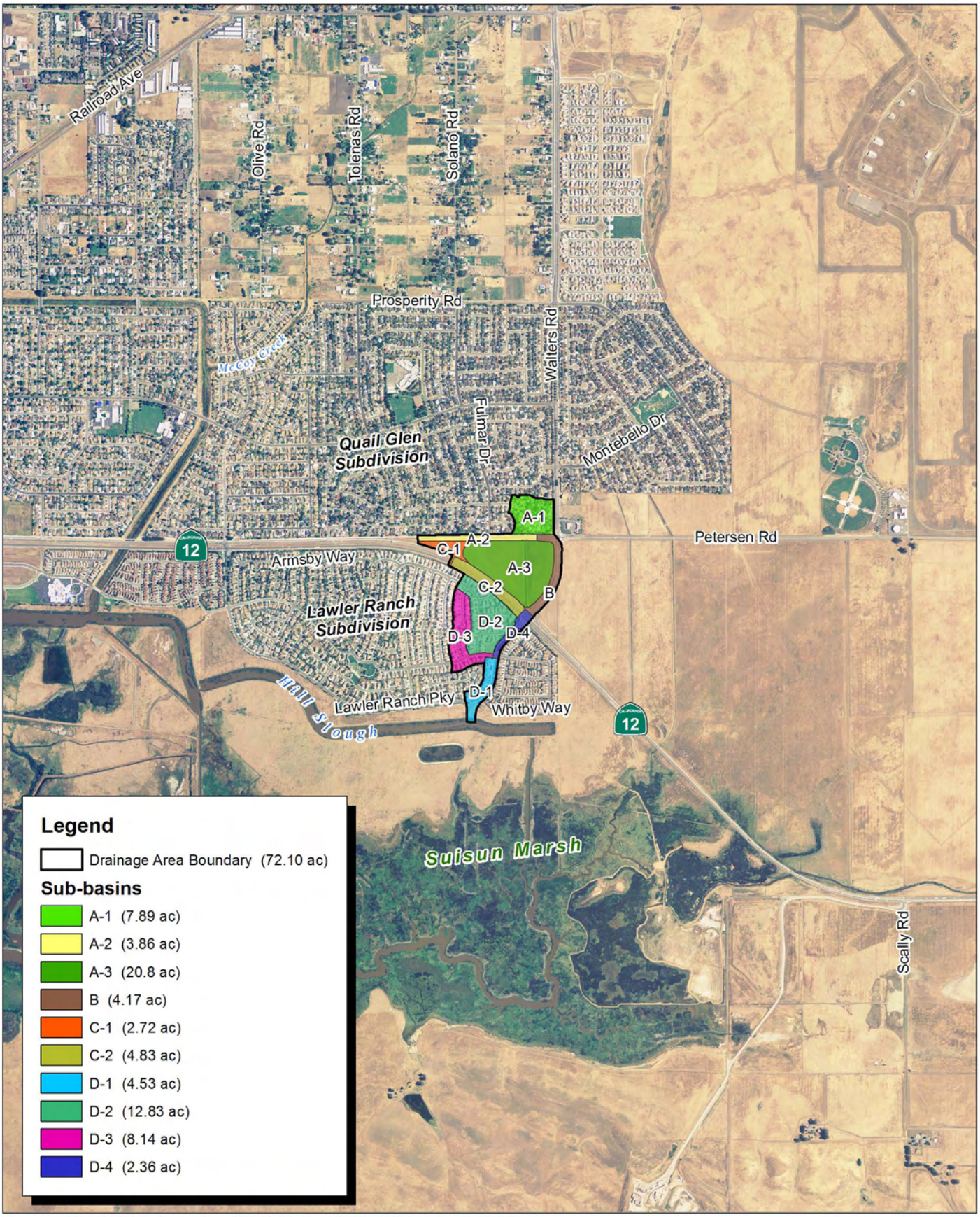
The rational formula is: $Q = CIA$

Where:

Q = peak runoff rate in cubic feet per second (cfs)
C = runoff coefficient, which is the ratio of the peak runoff rate to the average rainfall rate for a duration equal to the time of concentration
I = intensity of rainfall for a duration equal to the time of concentration (Tc) in inches per hour (in/hr)
A = drainage area in acres

For the purposes of this analysis, the Project area was divided into sub-areas (e.g. drainage catchments) and reaches (major flow paths within each drainage catchment). Each sub-area has a hydrograph generated from the land area based on the land and climate characteristics provided. Hydrographs from sub-areas and reaches are combined as needed to accumulate flow as water moves from the upland areas down through the watershed reach network. The accumulation of all runoff from the watershed is represented at the watershed outlet; in this instance the 52-inch outfall that discharges to Suisun Slough. The ± 72-acre drainage area was divided into four sub-areas with a total of 10 reaches modeled (see Exhibit 11).

All storm events have a certain probability of occurring within a given year. These probabilities are given the designation of a 2-, 5-, 10-, 25-, 50-, or 100-year storm. The reciprocal of the number of years is the probability of that particular storm occurring in one year (i.e., a 2-year storm would have a 50 percent probability of occurring in one year, etc.). The lower the probability of a storm to occur, the larger and more intense that storm will be when it does occur. This analysis and associated I values considers the 15, 25, and 100 year probabilities of storm events to get an idea of how the land base will respond to each type of precipitation volume and intensity with and without the Project.



Source: TOPO! USGS Denverton (1980), Elmira (1980), Fairfield South (1980), Fairfield North (1980); and MBA GIS (2007).

NORTH

2,000 1,000 0 2,000 Feet

Michael Brandman Associates

Exhibit 11
Drainage Sub-Basin Map
Aerial Base

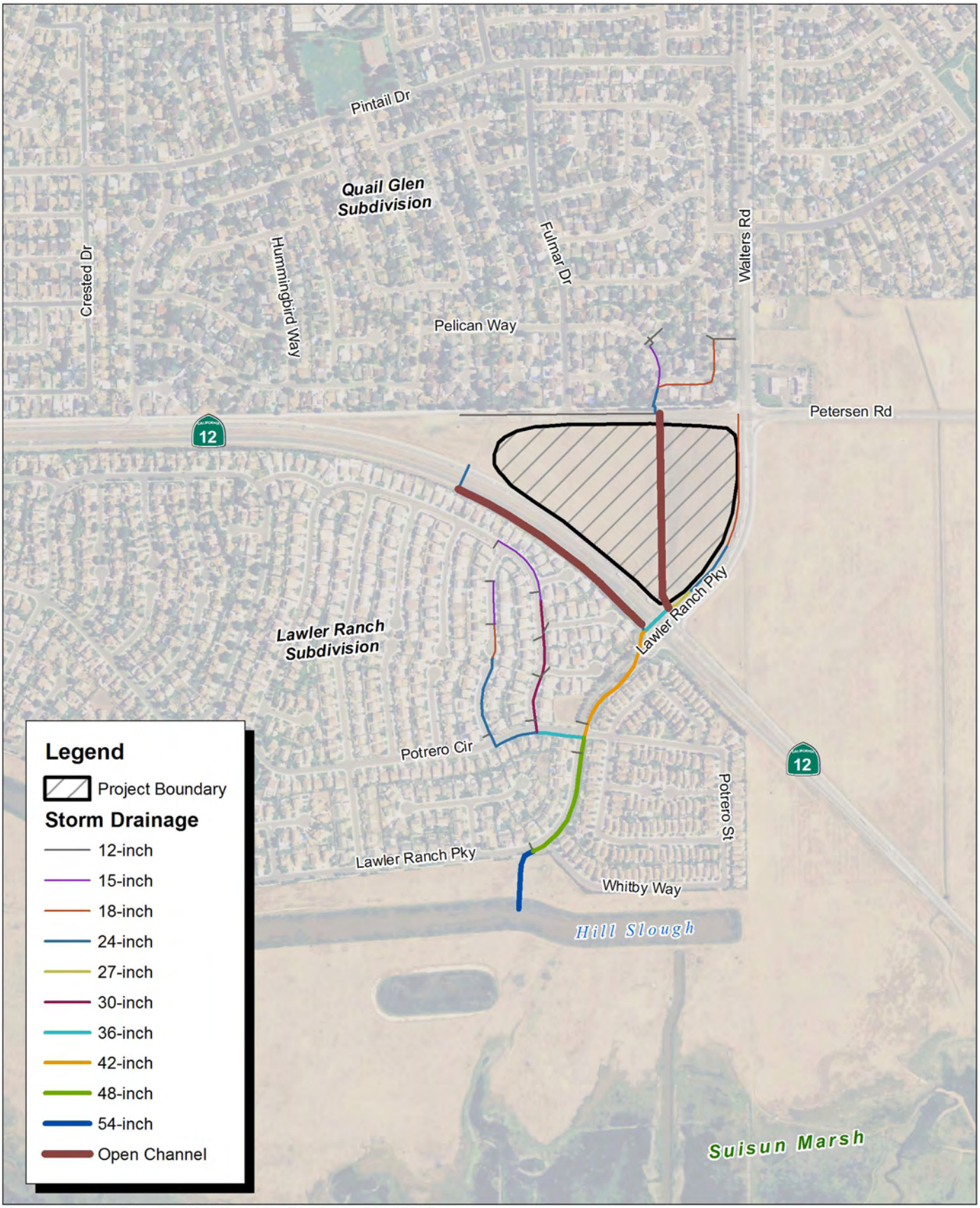
The runoff curve number, or *C*, is a number assigned to a surface that is related to the hydrologic soil group, but further considers the land use and the relative condition of a surface. The *C* value can range between 30 and 100. In general, high *C* values mean that a soil is in poor condition, more apt to infiltrate water slowly, and thus produces more of runoff. Low *C* values mean that a surface or soil is in good condition, more apt to infiltrate water quickly, and thus produces little runoff. MBA developed *C* values for each drainage subarea using Table 1 (Runoff Coefficients For Rational Method) of the City's Standards. Land uses were determined based on a review of the City's Zoning Map and a combination of aerial photograph interpretation and ground truthing for each of the modeled reaches shown in Exhibit 11.

Time of concentration (*Tc*) is the length of time (in minutes) water takes to flow from the most distant part of a drainage area to the outlet point. Per the City's Standards, *Tc* is derived from a combination of the overland flow travel time provided in Figure 1 of the City's Standards and the channel flow time to the design point. For sheet and shallow concentrated flow, the parameters of interest include the length, slope, and surface cover. The corresponding slope angle for sheet and concentrated flow was determined using assumed value(s) of one-half (0.5) and one percent (1.0) given a lack of any formal surveying in support of this study. A one percent slope angle is generally representative for much of the Project area topography and thus, provides a worst-case value for *Tc*. Reach lengths were determined by digitizing applicable linear drainage infrastructure, including open channels, through a combination of scanning and stitching of drainage engineering plans for the local subdivisions and ground truthing using global positioning systems (GPS). These digitized drainage features are illustrated in Exhibit 12. Surface roughness input values for the calculations are based on the *C* values for each basin and are generally limited to the conditions allowed in Table 1 of the Standards.

Flow routing was performed for the sub-basins that contain smaller sub-units and drain into one-another (e.g. sub-basins A1, A2 and A3). To route flows through a particular reach, data for the reach consisting of length, slope, and land cover are required. The reach length equals the channel or pipeline length to the down-slope receiving reach. Hydraulic capacity values for the existing downstream drainage system are based on data provided by the American Concrete Pipe Association. Computations for receiving reaches within each sub-basin are provided in Attachment A.

3.2 - Results and Discussion

After determining the *C* and *Tc* values for each sub-basin, MBA modeled storm water runoff scenarios consisting of the common 24-hour storms for the periods of 15, 25, and 100 years. Most Storm Water Control Plans (SCP) and many flood control strategies do not require designs for the 100-year storm, but modeling this scenario provides an upper limit for comparison and will also provide a conservative stability analysis for the elements of the SCP. The following model input was used to estimate total runoff for each sub-basin and sum total drainage within each of the four drainage sub-basins:



Source: City of Suisun City (2007), and MBA Field Studies (2007).

Michael Brandman Associates

NORTH

750 375 0 750 Feet

Exhibit 12
Drainage Infrastructure Aerial Base

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HYDROLOGIC & HYDRAULIC ANALYSIS

- Sub-Basin Area Units: acres
- Storm Data: see Table 2 of the City's Standards (provided in Attachment A)
- Sub-Area Land Use Details: values obtained from Table 1 of the City's Standards (provided in Attachment A)
- Time of Concentration: see Figure 1/Table 2 of the City's Standards - Travel Time For Overland Flow (provided in Attachment A)
- Results: see Table 1

The results of the model analysis (see Table 1) reveal the estimated rate of stormwater runoff (in cfs) for all storm events considered in the analysis. These are the absolute maximum water runoff rates occurring during a 24-hour storm. The results of this analysis assume structural routing throughout the Project drainage area. The calculations reflect two slope conditions (0.5 and 1 percent) to account for the potential coincidence of a high tide during one of the specified storm events, which could reasonably be expected to create backwater effect.

Table 1: Preliminary Drainage Calculations

Basin	Storm Event					
	15 year (cfs)		25 year (cfs)		100 Year (cfs)	
	Slope		Slope		Slope	
	0.5%	1%	0.5%	1%	0.5%	1%
Basin A1	4.19	4.90	5.62	6.42	9.13	10.42
Basin A2	7.67	8.37	11.12	8.54	14.59	13.30
Basin A3 (Existing Conditions)	7.57	8.57	10.15	11.31	16.33	18.72
Basin A3 (w/ Project)	7.94	31.47	11.97	45.40	15.71	86.94
Basin B	1.45	9.02	1.98	9.02	3.17	18.82
Basin C1	11.40	1.70	9.60	1.70	18.96	3.92
Basin C2	2.23	12.08	2.99	12.08	4.95	21.50
Basin D1	6.21	2.57	8.35	2.57	14.08	5.68
Basin D2	4.06	7.17	5.35	7.17	9.02	15.68
Basin D3	1.24	4.55	1.69	4.55	2.78	10.05
Basin D4	7.94	1.43	11.97	1.43	15.71	3.14
Total (Existing Conditions)	**53.95**	**60.35**	**68.82**	**74.53**	**108.71**	**121.21**
Total (w/ Project)	**74.13**	**83.25**	**102.18**	**83.25**	**170.18**	**189.44**
Net Increase from Project	**+ 20.19**	**+ 22.90**	**+ 33.36**	**+ 34.10**	**+ 61.46**	**+ 68.22**
Notes Supporting calculations are provided in Attachment A.						

Source: MBA, 2007; City of Fairfield Engineering Standards, 2006

Based on the modeling calculations, the Project would create a net increase of up to 22.90 cfs during a 15-year storm event when compared to existing conditions. Similarly, the net increase in peak runoff during 25-year and 100-year storm events is estimated up to 34.1 cfs and 68.2 cfs, respectively. Given that the Lawler Ranch trunk line receives flow from all up-gradient locations, the convergence location of the contributing storm drains just up-gradient of the trunk line (e. g. intersection of SR 12/Walters Road) is where capacity issues would be expected to arise.

The SCP for the Project is designed to provide a treatment capacity for onsite runoff of up to 49.79 cfs during a 15-year event and 50.56 cfs during a 25-year event. Based on the calculations provided in Table 1, these design capacities would be sufficient to treat onsite stormwater flows. Drainage flows originating from up-gradient locations would be routed underneath the site via a 36-inch pipe and would not necessitate additional treatment capacity. Onsite flows routed through the treatment system would connect to the 36-inch pipe near the southern corner of the Project site and contribute flow to the 42-inch Lawler Ranch Parkway trunk line.

Based on the drainage calculations for the Project site under existing conditions as provided in Table 1, it appears that the SCP would provide minimal attenuation of post-development stormwater flows. As previously indicated, the SCP includes a design capacity of up to 49.79 cfs during a 15-year event and 50.56 cfs during a 25-year event. However, it is unclear as to how the SCP would attenuate flows to pre-development levels following construction of the Project. As provided in Table 1, under existing conditions, the Project site (sub-basin A3) generates up to 8.57 cfs during a 15-year event, 11.31 during a 25-year event and 18.72 cfs during a 100-year event. The SCP provides no storage component that could reasonably be expected to accommodate a net difference of 22.90 cfs and 34.10 cfs between the pre- and post-Project conditions for the 15-year and 25-year storm events, respectively.

Based on this finding, it is reasonable to conclude that the Project would increase stormwater flows to downstream conveyance facilities. In addition to accommodating increased drainage flows from the developed Project site, the downstream drainage conveyance facilities within Lawler Ranch, would also need to continue to accommodate flows from sub-basins B and C (see Exhibit 12). The most widely accepted formula for evaluating the hydraulic capacity of non-pressure sewers is the Manning formula. The formula is:

$$Q = 1.486/n \times A \times R^{2/3} \times S^{1/2}$$

Where:

Q = discharge in cubic feet per second
n = Manning's roughness coefficient

A = cross-sectional area of flow in square feet
R = hydraulic radius in feet (equals the area of flow divided by the wetted perimeter)
S = slope of pipeline in feet of vertical drop per foot of vertical distance

The capacity of a given pipeline diameter and type is generally contingent on the design flow and pipe slope, with the Manning Formula more conveniently expressed as $Q/S^{1/2} = 1.486/n \times A \times R^{2/3}$. By evaluating the values of $1.486/n \times A \times R$ for various types and pipe diameters available, a pipe size can be selected for any $Q/S^{1/2}$ value. Under any given pipe flow condition, the area (A) and hydraulic radius (R) are constant for a particular size and shape of pipe. Therefore, the hydraulic capacity of a given pipe is primarily dependant on *n*, the roughness coefficient. For the purposes of this analysis, higher values of 0.012 and 0.013 are used to account for the build up of foreign debris in the storm sewers as observed in up-gradient sections of the storm drain system.

Table 2 provides the hydraulic capacity determinations for the downstream conveyance system based on each rainfall event and for both a 0.5 and 1.0 percent slope condition. Based on estimates provided by the American Concrete Pipe Association and using a conservative Manning's "n" value of 0.012, the 42-inch pipe would theoretically have enough capacity to contain flows up to the 25-year storm event, which would produce up to 87.46 cfs at the point of convergence. However, using the same methodology, the calculations indicate that the 100-year event, which would produce up to 1549.0 cfs, could overwhelm the conveyance system thereby resulting in on-street flooding.

Table 2: Down-Gradient Conveyance Capacity

Rainfall Event (Slope Condition)	Existing Conditions (Values for $1.486/n \times A \times R^{2/3}$)	Capacity Deficiency with Project?		W/ Project (Values for $1.486/n \times A \times R^{2/3}$)	Capacity Deficiency with Project?	
		Manning *n* value = 0.012 [1]	Manning *n* value = 0.013 [2]		Manning *n* value = 0.012 [1]	Manning *n* value = 0.013 [2]
1 % Slope Condition						
15 year (cfs)	446.4	No	No	675.4	No	No
25 year (cfs)	533.6	No	No	874.6	No	No
100 Year (cfs)	866.7	No	No	1549.0	Yes	Yes
0.5% Slope Condition						
15 year (cfs)	596.9	No	No	854.2	No	No
25 year (cfs)	713.2	No	No	1185.0	Yes	Yes
100 Year (cfs)	1101.5	Yes	Yes	1970.7	Yes	Yes

1. The maximum conveyance capacity for a 42-inch diameter concrete pipe with a manning surface roughness value of 0.012 in 1090 cfs.

2. The maximum conveyance capacity for a 42-inch diameter concrete pipe with a manning surface roughness value of 0.013 in 1006 cfs.

Source. Tables II and III, Design Data 4, Hydraulic Capacity of Sewers, American Concrete Pipe Association,

Further, if the pipe slope were reduced to an effective gradient of 0.5 percent in response to periods of high tide or long-term incremental increases in sea level, the existing downstream conveyance system's capacity would be compromised at the 25-year event (see Table 2). Based on these findings, it is reasonable to conclude that flows at this convergence point could result in significant capacity reductions in the Lawler Ranch drainage system thereby resulting in flooding within portions of the Lawler Ranch subdivision.

Based on MBA's preliminary findings, MBA has developed the following recommendations to address potential impacts to the existing drainage system:

- Drainage conveyance capacity within the downstream drainage system is limited and would be overwhelmed during a 100-year storm event with the addition of the Project. This situation would be worsened by a reduction in the pipeline's effective vertical slope in response to either a high tidal event during the specified rainfall event or by incremental increases in long-term sea level. To minimize potential flooding impacts to downstream residents, the Project will be required to implement one of two options: (1) increase the diameter of the downstream drainage conveyance line, or (2) provide sufficient on-site storage to retain peak flows to an acceptable level. If on-site storage is preferred, the storage facility should provide sufficient capacity to contain the net difference between the hydraulic capacity of the existing conveyance line and the flow values determined by the equation $1.486/n \times A \times R^{2/3}$ as provided for each storm event.

- The City should investigate the condition of the downstream conveyance systems, as no visual inspection of the conveyance facilities was conducted as part of the study. If observations indicate that restrictions in conveyance capacity are occurring, the City should have the downstream conveyance system flushed to maximize the existing drainage capacity.

- Onsite detention could be provided via several means including temporary storage within the parking lot, an under-ground vault, a linear retention facility along the project site's southern and/or eastern perimeter, or acquiring the vacant lot adjacent and to the west for the construction of a detention/retention facility.

3.3 - Limitations

This analysis provides a preliminary estimate of runoff volumes from sections of the Project area that would be affected by the Project and allows planners to evaluate the effects of land use changes associated with the Project that may affect runoff volume. The results of the modeling effort are considered estimates based on the conditions and assumptions input into the model, and are used to gauge the relative overall impact of storm water drainage on existing development. The results, therefore, are not considered to be final design flows and are not intended for project-specific endeavors.

SECTION 4: IMPLICATIONS TO WATER QUALITY

4.1 - Water Quality Regulations

The San Francisco RWQCB has developed a Water Quality Control Plan (Basin Plan) for the San Francisco Bay region (San Francisco RWQCB, 1995). The Basin Plan contains water quality objectives designed to "define appropriate levels of environmental quality and control activities that can adversely affect aquatic systems." The RWQCB has defined beneficial uses as a basis for establishing water quality standards and discharge prohibitions. The following existing beneficial uses are listed for Suisun Bay and would apply to Grizzly Bay and Suisun Slough.

- **Ocean, Commercial, and Sport Fishing.** This includes uses of water for commercial or recreational collection of fish, shellfish, or other organisms in oceans, bays, and estuaries, including, but not limited to, uses involving organisms intended for human consumption or bait purposes. (Suisun Bay only)
- **Estuarine Habitat.** This includes uses of water that support estuarine ecosystems, including, but not limited to, preservation or enhancement of estuarine habitats, vegetation, fish, shellfish, or wildlife (e.g., estuarine mammals, waterfowl, shorebirds), and the propagation, sustenance, and migration of estuarine organisms. (Suisun Bay only).
- **Industrial Service Supply.** This includes uses of water for industrial activities that do not depend primarily on water quality, including, but not limited to, mining, cooling water supply, hydraulic conveyance, gravel washing, fire protection, and oil well repressurization. (Suisun Bay only)
- **Industrial Process Supply.** Uses of water for industrial activities that depend primarily on water quality. (Suisun Bay only)
- **Fish Migration.** This includes uses of water that support habitats necessary for migration, acclimatization between freshwater and saltwater, and protection of aquatic organisms that are temporary inhabitants of waters within the region. (Suisun Bay only)
- **Navigation.** This includes uses of water for shipping, travel, or other transportation by private, military, or commercial vessels.
- **Preservation of Rare and Endangered Species.** This includes uses of waters that support habitat necessary for the survival and successful maintenance of plant or animal species established under state and/or federal law as rare, threatened, or endangered. (Suisun Bay only)
- **Water Contact Recreation.** This includes uses of water for recreational activities involving body contact with water where ingestion of water is reasonably possible.

- **Noncontact Recreation.** This includes uses of water for recreational activities involving proximity to water but not normally involving contact with water where ingestion is reasonably possible.
- **Fish Spawning.** This includes uses of water that support high quality aquatic habitats suitable for reproduction and early development of fish.
- **Warm Freshwater Habitat.** This includes uses of water that support warm water ecosystems including, but not limited to, preservation or enhancement of aquatic habitats, vegetation, fish, or wildlife, including invertebrates. (Suisun Slough only)
- **Wildlife Habitat.** This includes uses of waters that support wildlife habitats, including, but not limited to, the preservation and enhancement of vegetation and prey species used by wildlife, such as waterfowl.

Based on the applied beneficial uses, the RWQCB has set water quality objectives for all surface waters in the Bay Area. These water quality objectives include bacteria, biostimulatory substances, chemical constituents, color, dissolved oxygen, floating material, oil and grease, pH, pesticides, radioactivity, salinity, sediment, settleable material, suspended material, sulfide, tastes and odors, temperature, toxicity, and turbidity (RWQCB, 1995). In addition, objectives for specific chemical constituents have been set depending on the beneficial uses designated for each waterbody (RWQCB, 1995). Constituent limits identified in the Basin Plan in conjunction with California Toxics Rule (CTR) criteria were used to determine the magnitude of any water quality impairment that could result from the Project.

In addition to standards and objectives identified in the Basin Plan, Provision B of Order No. R2-2003-0034 outlines receiving water limitations. Provision B1 specifically requires that a discharge shall not cause the following conditions to create a condition of nuisance or to adversely affect beneficial uses of waters of the State:

a) Floating, suspended, or deposited macroscopic particulate matter, or foam;

b) Bottom deposits or aquatic growths;

c) Alteration of temperature, turbidity, or apparent color beyond present natural background levels;

d) Visible, floating, suspended, or deposited oil or other products of petroleum origin; and/or

e) Substances present in concentrations or quantities which will cause deleterious effects on aquatic biota, wildlife, or waterfowl, or which render any of these unfit for human consumption.

4.2 - Urban Water Quality

The quality and quantity of runoff discharges from the Project area varies considerably and is affected by local hydrology, season, and the sequence and duration of individual hydrologic events. Pollutants of concern in these discharges include certain heavy metals, excessive sediment production from erosion, petroleum hydrocarbons from sources such as motor oil, certain pesticides associated with the risk of acute aquatic toxicity, excessive nutrient loads, and trash. Many of these pollutants are pervasive in urban environments and pollutants such as trash and excessive sediment were observed in local drainage ways as shown in Photo B of Exhibit 6.

No water quality data was acquired as part of this Study and, therefore, no site-specific data is available to characterize surface water quality for the Project area. However, based on numerous studies conducted by the U.S. EPA to characterize the nature of urban stormwater runoff, including the National Urban Runoff Program (NURP); the USGS Urban Stormwater Database; and the Federal Highway Administration (FHWA) study of stormwater runoff loadings from highways, sufficient data exists to characterize the basic nature of stormwater discharges based on land use. The most comprehensive study of urban runoff was the NURP, conducted by the EPA between 1978 and 1983. NURP focused on the following ten constituents:

- Total Suspended Solids (TSS)
- Biochemical Oxygen Demand (BOD)
- Chemical Oxygen Demand (COD)
- Total Phosphorus (TP)
- Soluble Phosphorus (SP)
- Total Kjeldahl Nitrogen (TKN)
- Nitrate + Nitrite (N)
- Total Copper (Cu)
- Total Lead (Pb)
- Total Zinc (Zn)

Since the NURP study, other important studies have been conducted to characterize stormwater. The University of Alabama and the Center for Watershed Protection were awarded an EPA Office of Water 104(b)3 grant in 2001 to collect and evaluate stormwater data from a representative number of NPDES municipal separate storm sewer system (MS4) stormwater permit holders. As of September 2003, these agencies have gathered data from 3,770 separate storm events from 66 agencies and municipalities from 17 states and developed the National Stormwater Quality Database (NSQD). A major goal of the NSQD project was to provide a benchmark for comparison with locally collected data. The NSQD provides typical values for associated land use classes. Preliminary data results for the NSQD are included in Table .

Table 3: Median Values and Event Median Concentrations for Selected Parameters in the NSDQ, Version 1.

Parameter	Overall	Residential	Commercial	Freeways	Open Space
Area (acres)	56	57.3	38.8	1.6	73.5
% Imperv.	54.3	37	83	80	2
Precip. Depth (in)	0.47	0.46	0.39	0.54	0.48
TSS (mg/L)	58	48	43	99	51
BOD (mg/L)	8.6	9	11.9	8	4.2
COD (mg/L)	53	55	63	100	21
Fecal Coliform MPN/100mL)	5,081	7,750	4,500	1,700	3,100
NH3 (mg/L)	0.44	0.31	0.5	1.07	0.3
NO2+NO3 (mg/L)	0.6	0.6	0.6	0.3	0.6
Nitrogen, Total Kjeldahl (mg/L)	1.4	1.4	1.6	2	0.6
Phos., total (mg/L)	0.27	0.3	0.22	0.25	0.25
Cd, total (ug/L)	1	0.5	0.9	1	0.5
Cu, total (ug/L)	16	12	17	35	5.3
Pb, total (ug/L)	16	12	18	25	5
Ni, total (ug/L)	8	5.4	7	9	ND
Zn, total (ug/L)	116	73	150	200	39

ND = not detected, or insufficient data to present as a median value.

Source: Center for Watershed Protection, 2004

4.3 - Water Quality BMP Evaluation and Discussion

MBA has determined that the Project will be subject to the numeric sizing criteria as provided in Provision C.3.d of Order No. R2-2003-0034. A review of the Project SCP indicates that many of the recommendations are already included. In addition, language provided in Order No. R2-2003-0034 suggests that the Project would not be subject to Provision C.3.f. of the Permit, which requires the preparation of a hydrograph modification management plan for areas. The basis for this conclusion is supported by Provision C.3.f(ii) which states “this requirement does not apply to new development projects where the project discharges stormwater runoff into the downstream sections of Laurel and Ledgewood Creeks or storm drains where the potential for erosion, or other related impacts to beneficial uses, is minimal. Such situations may include discharges into creeks that are concrete-lined or significantly hardened (e.g., with rip-rap, sackrete, etc.) downstream to their outfall in Suisun Marsh, underground storm drains discharging to Suisun Bay, and construction of infill projects in highly developed watersheds, where the potential for single-project and/or cumulative impacts is

minimal." Given that the Project would discharge into an underground storm drain that discharges directly into Suisun Bay, this exception would apply to the Project.

The Project utilizes a flow hydraulic design basis for onsite treatment measures whose primary mode of pollutant removal depends on flow capacity, such as swales, sand filters, or wetlands. Order R2-2003-0034 requires that these best management practices (BMPs) be sized to treat:

1. Ten percent of the 50-year peak flow rate; or
2. The flow of runoff produced by a rain event equal to at least two times the 85th percentile hourly rainfall intensity for the applicable area, based on historical records of hourly rainfall depths; or
3. The flow of runoff resulting from a rain event equal to at least 0.2 inches per hour intensity.

Infiltration BMPs are designed to capture a volume of stormwater runoff, retain it and infiltrate that volume into the ground. The advantages of infiltration is that it reduces the volume of water that is discharged to downstream conveyance facilities, thereby reducing some of the potential impacts caused by excess flows as well as increased pollutant concentrations in the receiving stream. Infiltration systems can be designed to capture a volume of storm water and infiltrate this water into the ground over a period of several hours or even days, thereby maximizing the infiltrative capacity of the BMP. Pollutant removal occurs as water percolates through the various soil layers. As the water moves through the soil, pollutants adhere to the soil particles. In addition, microorganisms in the soil can degrade organic pollutants that are contained in the infiltrated stormwater.

Although infiltration of stormwater has many benefits, it also has some drawbacks. First, the performance of infiltration BMPs is limited in areas with poorly permeable soils, such as the Project area. For this reason, the design engineer must verify soil permeability. A percolation rate of 0.5 inches per hour or more and a soil depth of 4 feet or more above groundwater are critical for success. In addition, infiltration BMPs can experience reduced infiltrative capacity and even clogging due to excessive sediment accumulation, thereby potentially requiring frequent maintenance to restore the infiltrative capacity of the system. Many failures can also be attributed to contractor inexperience and to improper design and siting (EPA, 1999).

Table summarizes the available field data on the efficiency of infiltration practices in treating stormwater. Reported removal efficiencies are based on the results of three studies that evaluated the performance of infiltration trenches and two studies that evaluated the efficiency of porous pavement systems (EPA, 1999). There is little data available, however, regarding the potential mobility of metals and hydrocarbons that enter groundwater due to infiltration of stormwater. This may be a particular problem in areas with extremely high soil permeabilities where pollutants can rapidly enter underlying aquifers with insufficient contact time for breakdown or adsorption of contaminants.

Table 4: BMP Expected Pollutant Removal Efficiency

Structural BMP Type	Typical Pollutant Removal (percent)				
	Suspended Solids	Nitrogen	Phosphorus	Pathogens	Metals
Infiltration Basins	50 – 80	50 – 80	50 – 80	65 – 100	50 – 80
Infiltration Trenches / Dry Wells	50 – 80	50 – 80	15 – 45	65 – 100	50 – 80
Porous Pavement	65 – 100	65 – 100	30 – 65	65 – 100	65 – 100
Grassed Swales	30 – 65	15 – 45	15 – 45	<30	15 – 45
Vegetated Filter Strips	50 – 80	50 – 80	50 – 80	<30	50 – 80
Source: EPA, 1999; EPA, 1993.					

Based on the potential water quality pollutants generated by commercial land uses as provided in Table 3 and the removal efficiencies provided in Table , it is reasonable to conclude the proposed infiltration systems my not sufficiently treat heavy metal loadings, nutrients, and would be susceptible to clogging from excess sediment. In addition, once the treatment capacity of the infiltration system is reached, drainage flows would bypass the infiltration systems and flow directly into the storm drainage system with no pretreatment. Further, based on the poor soil drainage conditions exhibited onsite, the feasibility of long-term infiltration may be questionable. The City should require that the Applicant conduct soil percolation testing onsite to confirm the feasibility of infiltration BMPs to confirm that the necessary infiltration capacity exists and provide supplemental BMPs, as necessary, to address those pollutants not otherwise effectively treated by the methods proposed.

In moving forward, the design of Project water quality BMPs in the SCP should employ three basic strategies in the following order of relative effectiveness: (1) reduce or eliminate post-project runoff; (2) identify target pollutants and emphasize source control, and (3) treat stormwater runoff in advance of discharging it to the drainage system and Suisun Marsh. Beyond additional measures recommended in this Report to minimize post-Project drainage flows in conjunction with infiltration BMPs proposed in the Project SCP, source reduction practices will be essential in controlling the amounts of pollutants entering stormwater runoff. Source reduction practices minimize the introduction of urban contaminants into the landscape thereby limiting the amounts of pollutants contained in stormwater runoff following a given rainfall event. Examples of source reduction include limiting applications of fertilizers, pesticides and herbicides; periodic street sweeping to remove trash, litter and particulates from streets; collection and disposal of lawn debris; periodic cleaning of catch basins; and the elimination of improper dumping practices for used oil, antifreeze, household cleaners, paint, etc.

In taking into account the minimum expected pollutant removal efficiencies of the vegetated swale, infiltration planter, or proprietary device as proposed, the Applicant should select a combination of additional BMPs that, given the lowest expected pollutant removal efficiencies provided in Table 4,

will cumulatively remove as close to a 100 percent of the target pollutants as possible. Target pollutants for this Project include pathogens, heavy metals, nutrients, pesticides, organic compounds, suspended solids and sediment, trash and debris, oxygen demanding substances, and oil and grease. BMPs available for inclusion in the project prior to finalization of the project's design may include a combination of the following stormwater treatment devices:

- Retention/Detention Ponds
- Retention Rooftops
- Green roofs (roofs that incorporate vegetation) and blue roofs (roofs that incorporate detention or retention of rain).
- Porous/Permeable Pavement
- Crushed stone reservoir base rock under pavements or in sumps
- Oil/Grease Separators
- Compost Berms
- Street Sweeping

SECTION 5: REFERENCES

Bay Area Regional Water Quality Control Board (RWQCB). 1995. The Water Quality Control Plan (Basin Plan) for the California Regional Water Quality Control Board Bay Area Region. Fourth edition.

Bay Area Regional Water Quality Control Board, 2003. Fairfield-Suisun Areawide NPDES Municipal Stormwater Permit, Order No. R2-2003-0034, NPDES Permit No. CAS612005

Calwater 2.2.1, 1999. California Interagency Watershed Map of 1999 (CalWater Version 2.2.1),

California Geological Survey, 2002. Note 36, California Geomorphic Provinces

CDEC, 2007. California Data Exchange Center. Tidal Data for Suisun Slough. February, 2007

Center for Watershed Protection, 2004. Findings from the National Stormwater Quality Database (NSQD). Prepared by Robert Pitt, Alex Maestre, and Renee Morquecho Dept. of Civil and Environmental Engineering, University of Alabama and Ted Brown, Tom Schueler, Karen Cappiella, Paul Sturm, and Chris Swann, Center for Watershed Protection. January 2004

City of Fairfield, 2006. Engineering Standards and Specifications (Standards). Section 4 Storm Drainage

City of Suisun City, 1992. City of Suisun City General Plan, Land Use Element and Map. Adopted May 1992.

EPA (U.S. Environmental Protection Agency), 1993. Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters, EPA-840-B-92-002.

EPA (U.S. Environmental Protection Agency), 1999. Preliminary Data Summary of Urban Storm Water Best Management Practices, EPA-821-R-99-012, August 1999.

EPA, 1983. Environmental Protection Agency, Dec. Results of the Nationwide Urban Runoff Program (NURP). Water Planning Division, PB 84-185552, Washington, D.C. 1983.

Fairfield-Suisun Sewer District and the cities of Fairfield and Suisun City which have joined together to form the Fairfield-Suisun Urban Runoff Management Program.

Federal Emergency Management Agency (FEMA), 2007. Flood Insurance Rate Map

Lawler Ranch Drainage Infrastructure Map, 1989. The Lawler Ranch, Unit 3, Suisun City, Ca. Prepared by Jack Johnson Engineers, Inc. 1989

National Oceanic and Atmospheric Administration, Western U.S. Precipitation Frequency Maps, NOAA Atlas 2, 1973. 15

National Pollutant Discharge Elimination System (NPDES) Permit No. CAS612005 issued under Order No. 95-079 on April 19, 1995

NRCS, 1977. Natural Resources Conservation Service, Soil Survey for Solano County, California (1977)

Robert A. Karn and Associates, 2006. Preliminary Hydrologic and Hydraulic Calculations and Stormwater Control Plan for the Walmart Store, September 8, 2006.

Solano County Hydrology Manual, 1999. Prepared for the Solano County Water Agency, Prepared by West Yost and Associates. June 1999. http://www.scwa2.com/hydrology%20manual.html

SWRCB (State Water Resources Control Board). Policy for the Implementation of Toxic Standards for Inland Surface Waters, Enclosed Bays and Estuaries of California. Resolution 2000-015, 2000.

SWRCB. Revision of the Clean Water Act Section 303(d) List of Water Quality Limited Segments, Draft Staff Report. State Water Resources Control Board Division of Water Quality. April 2,

U.S. Environmental Protection Agency. 2003. *2002 California Section 303(d) List and TMDL Priority Schedule.*

USGS, 2004. Water Resources Data, California, Water Year 2004, Volume 2. Pacific Slope Basins from Arroyo Grande to Oregon State Line except Central Valley. Water-Data Report CA-04-2. U.S. Department of the Interior, U.S. Geological Survey Water Resources Data

USGS, 2007. Geologic Map of the San Francisco Bay Region, U.S. Department of the Interior, U.S. Geological Survey; http://geomaps.wr.usgs.gov/sfgeo/geologic/downloads.html

Western Regional Climate Center, 2000. Western U.S. Precipitation Frequency Maps. Online resource: http://www.wrcc.dri.edu/pcpnfreq.html. Accessed February 5, 2007.

Attachment A: Drainage Calculations

TABLE 1

RUNOFF COEFFICIENTS FOR RATIONAL METHOD

SURFACE OR AREA TYPE	RUNOFF COEFFICIENT, C
Paved areas (asphalt or concrete)	0.95
Industrial areas	0.85
Commercial areas	0.85
Residential areas	
Single family, avg. slope less than 2%	0.50
Single family, avg. slope between 2% and 7%	0.55
Single family, avg. slope greater than 7%	0.65
Multi-family, detached	0.65
Multi-family, attached	0.70
Schools	0.45
Agricultural land	0.45
Undeveloped open spaces, including pasture	
Average slope less than 2%	0.40
Average slope between 2% and 7%	0.47
Average slope greater than 7%	0.55
Oak timber and heavy brush	
Average slope less than 2%	0.35
Average slope between 2% and 7%	0.42
Average slope greater than 7%	0.50

These coefficients are to be used for a return period of 15 years. For return periods of 25 and 100 years, modify the table values as follows:

25 YEAR RETURN: C = TABLE VALUE X (1.07)
100 YEAR RETURN: C = TABLE VALUE X (1.25)

NOTE: No value of "C" shall be modified beyond 1.0.

TABLE 2

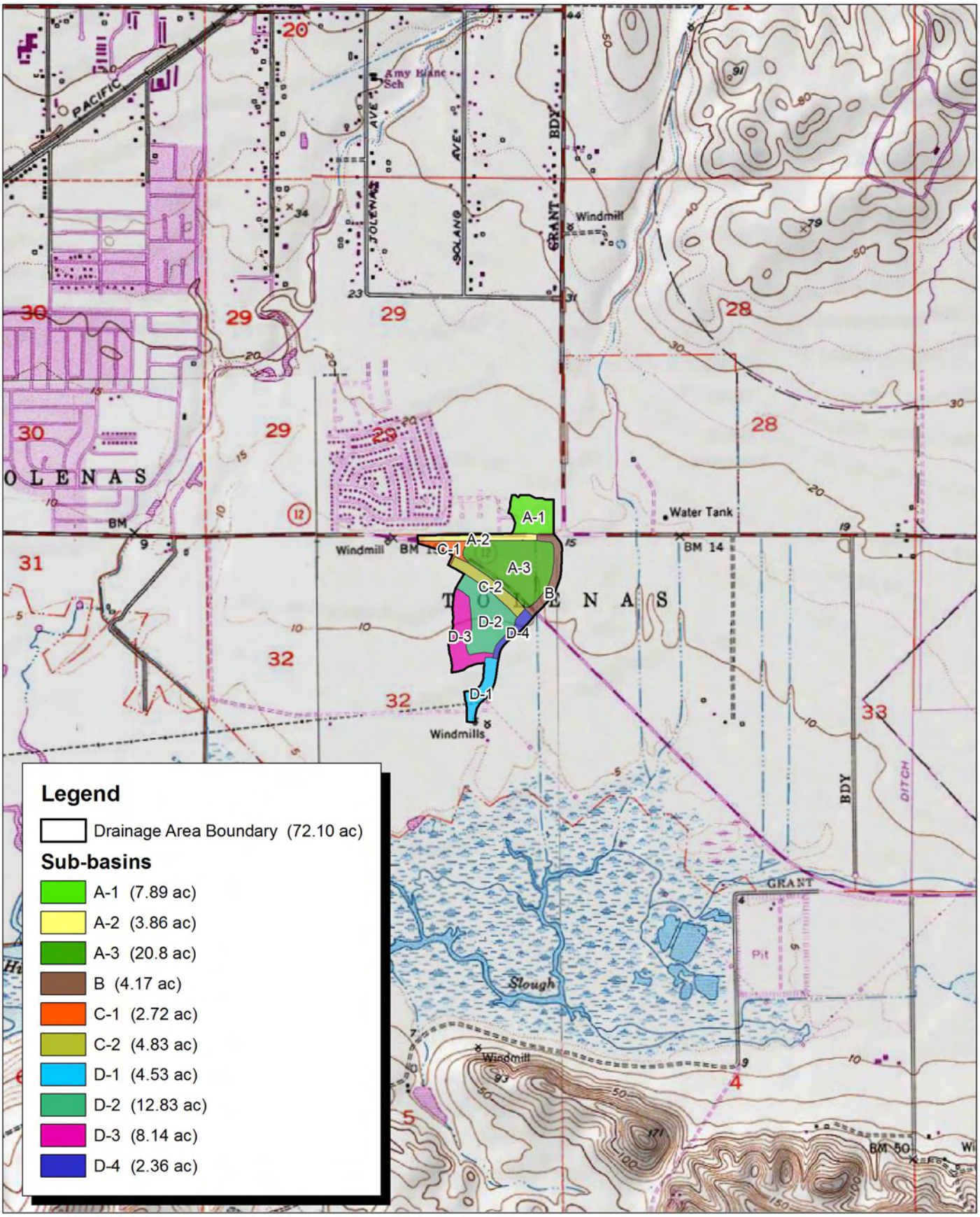
TIME OF CONCENTRATION IN MINUTES	INTENSITY OF RAINFALL IN INCHES PER HR.			TIME OF CONCENTRATION IN MINUTES	INTENSITY OF RAINFALL IN INCHES PER HR.		
	15YR	25YR	100YR		15YR	25YR	100YR
5	3.13	3.76	4.93	36	1.16	1.40	1.82
6	2.84	3.41	4.48	37	1.14	1.38	1.80
7	2.65	2.19	4.18	38	1.12	1.34	1.76
8	2.50	2.00	3.95	39	1.11	1.33	1.74
9	2.37	2.85	3.74	40	1.09	1.32	1.72
10	2.26	2.71	3.53	41	1.08	1.30	1.70
11	2.16	2.60	3.38	42	1.06	1.28	1.67
12	2.07	2.49	3.25	43	1.05	1.27	1.66
13	1.99	2.40	3.14	44	1.03	1.24	1.62
14	1.91	2.30	3.01	45	1.02	1.22	1.61
15	1.84	2.21	2.90	46	1.01	1.21	1.60
16	1.78	2.15	2.81	47	1.00	1.20	1.57
17	1.72	2.07	2.71	48	0.99	1.19	1.56
18	1.67	2.01	2.64	49	0.97	1.17	1.54
19	1.62	1.95	2.52	50	0.96	1.16	1.52
20	1.57	1.88	2.47	51	0.95	1.15	1.50
21	1.53	1.84	2.41	52	0.94	1.14	1.48
22	1.49	1.79	2.35	53	0.93	1.13	1.46
23	1.45	1.74	2.29	54	0.92	1.12	1.45
24	1.42	1.71	2.24	55	0.91	1.10	1.44
25	1.39	1.67	2.20	56	0.90	1.09	1.43
26	1.36	1.64	2.15	57	0.89	1.07	1.42
27	1.34	1.61	2.11	58	0.89	1.06	1.41
28	1.32	1.58	2.09	59	0.88	1.05	1.40
29	1.30	1.56	2.05	60	0.87	1.04	1.38
30	1.28	1.54	2.02	70	0.80	0.96	1.26
31	1.26	1.52	1.99	80	0.74	0.89	1.16
32	1.24	1.50	1.96	90	0.69	0.83	1.09
33	1.22	1.46	1.92	100	0.65	0.78	1.02
34	1.19	1.43	1.87	110	0.62	0.75	0.98
35	1.17	1.41	1.85	120	0.59	0.70	0.94

Roof-to-gutter time shall be taken as 9 minutes for residential areas and 5 minutes toon-site facilities for commercial and industrial sites.The time of concentration shall be derived from a combination of the overland flow travel time from Figure 1, and the channel flow time to the design point.

Rainfall intensities obtained from this table are based on a mean annual precipitation of 21 inches. To correct these values for areas with different mean annual precipitation (MAP), apply the following correction to the tabulated intensities:

corrected intensity = (table value) x (actual MAP) / (21)

	Existing Condtions				W/ Project			
	15 year	*25 year*	*100 year*		*15 year*	*25 year*	*100 year*	
Basin A1	4.90	6.42	10.42	*cfs*	4.90	6.42	10.42	*cfs*
Basin A2	8.37	8.54	13.30	*cfs*	8.37	8.54	13.30	*cfs*
Basin A3	8.57	11.31	18.72	*cfs*	31.47	45.40	86.94	*cfs*
Basin B	9.02	8.40	18.82	*cfs*	9.02	8.40	18.82	*cfs*
Basin C1	1.70	2.32	3.92	*cfs*	1.70	2.32	3.92	*cfs*
Basin C2	12.08	16.37	21.50	*cfs*	12.08	16.37	21.50	*cfs*
Basin D1	2.57	3.44	5.68	*cfs*	2.57	3.44	5.68	*cfs*
Basin D2	7.17	9.66	15.68	*cfs*	7.17	9.66	15.68	*cfs*
Basin D3	4.55	6.14	10.05	*cfs*	4.55	6.14	10.05	*cfs*
Basin D4	1.43	1.93	3.14	*cfs*	1.43	1.93	3.14	*cfs*
Total/Outlet	60.35	74.53	121.21	*cfs*	83.25	108.62	189.44	*cfs*
Net Difference	**22.90**	**34.10**	**68.22**					
Convergance SR12/Walters Road	44.64	53.36	86.67		67.54	87.46	154.90	
net	**22.90**	**34.10**	**68.22**					



Source: TOPO! USGS Denverton (1980), Elmira (1980), Fairfield South (1980), Fairfield North (1980); and MBA GIS (2007).

NORTH

2,000 1,000 0 2,000 Feet

Michael Brandman Associates

Exhibit 11
Drainage Sub-Basin Map
Topographic Base

Sub Area A1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	4.898	Qp	6.42428	Qp	10.41813
C	0.5	C	0.535	C	0.625
I	1.24	I	1.52	I	2.11
A	7.9	A	7.9	A	7.9

Rational Method

Qp=CIA

Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 1 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Inputs

Sub Area A2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	8.3733	Qp	8.541	Qp	13.299
C	0.95	C	1	C	1
I	2.26	I	2.19	I	3.41
A	3.9	A	3.9	A	3.9

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	paved area = 0.95
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area A3

Existing Conditions 15 year		Existing Conditions 25 year	(15yr C * 1.2	Existing Conditions 100 yea	(15yr C * 1.2	w/ Project 15 year		w/ Project 25 year	(15yr C * 1.25)	w/ Project 100 year	(15yr C * 1.25)
Qp	8.5696	Qp	11.306	Qp	18.72	Qp	31.4704	Qp	45.40224	Qp	86.944
C	0.4	C	0.428	C	0.5	C	0.85	C	0.9095	C	1
I	1.03	I	1.27	I	1.8	I	1.78	I	2.4	I	4.18
A	20.8	A	20.8	A	20.8	A	20.8	A	20.8	A	20.8

Rational Method

Qp=CIA

Inputs

Qp	peak flow rate (cfs)	
C	runoff coefficient (dimensional Table 1	Existing Conditions - Undeveloped Open Space, including pasture > 2 % slope = 0.4
I	rainfall intensity (in/hr)	W/ Project - Commerical = 0.85
A	drainage area (aces)	Figure 1/Table 2 - Travel Time For Overland Flow

Sub Area B

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	9.0174	Qp	8.4	Qp	18.816
C	0.95	C	1	C	1
I	2.26	I	2	I	4.48
A	4.2	A	4.2	A	4.2

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	paved area = 0.95
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area C1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	1.6956	Qp	2.322756	Qp	3.915
C	0.4	C	0.428	C	0.5
I	1.57	I	2.01	I	2.9
A	2.7	A	2.7	A	2.7

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	Undeveloped Open Space, including pasture > 2 % slope = 0.4
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area C2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	12.084	Qp	16.368	Qp	21.504

Note: Subbasin C2 was subdivided into two smaller basins (C2a and C2b) due to the a drainage length that is > 1200 ft.

Rational Method

Qp=CIA			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	paved area = 0.95
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)		(C2a)
Qp	6.042	Qp	8.184	Qp	10.752	
C	0.95	C	1	C	1	
I	2.65	I	3.41	I	4.48	
A	2.4	A	2.4	A	2.4	

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)		(C2b)
Qp	6.042	Qp	8.184	Qp	10.752	
C	0.95	C	1	C	1	
I	2.65	I	3.41	I	4.48	
A	2.4	A	2.4	A	2.4	

Sub Area D1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	2.565	Qp	3.442725	Qp	5.68125
C	0.5	C	0.535	C	0.625
I	1.14	I	1.43	I	2.02
A	4.5	A	4.5	A	4.5

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area D2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	7.168	Qp	9.65568	Qp	15.68
C	0.5	C	0.535	C	0.625
I	1.12	I	1.41	I	1.96
A	12.8	A	12.8	A	12.8

Rational Method

Qp=CIA			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area D3

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	4.551	Qp	6.1418	Qp	10.045
C	0.5	C	0.535	C	0.625
I	1.11	I	1.4	I	1.96
A	8.2	A	8.2	A	8.2

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

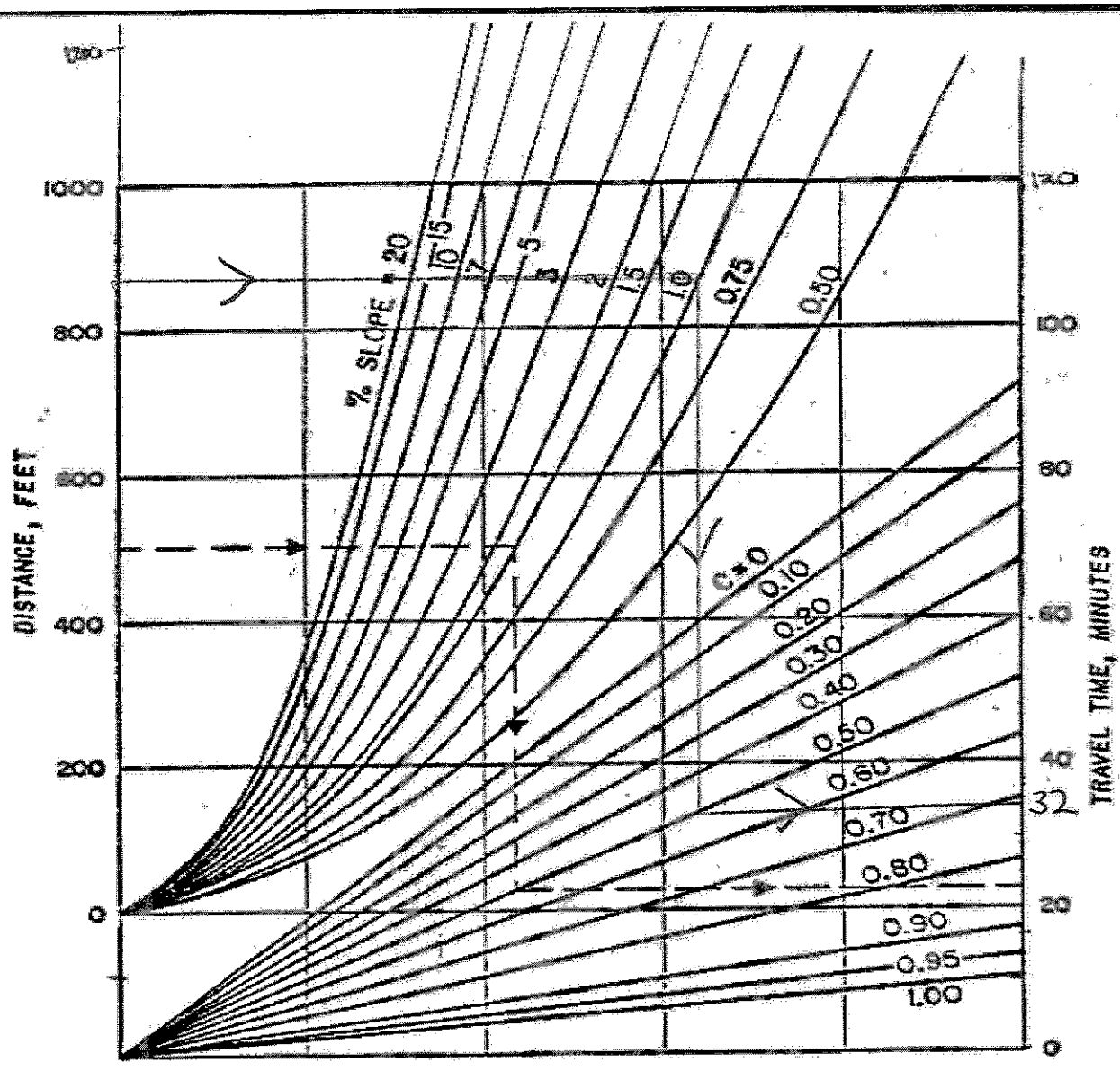
Sub Area D4

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	1.428	Qp	1.926	Qp	3.135
C	0.5	C	0.535	C	0.625
I	1.19	I	1.5	I	2.09
A	2.4	A	2.4	A	2.4

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		



A-1 (15 yr.)

Length = 865 ft.

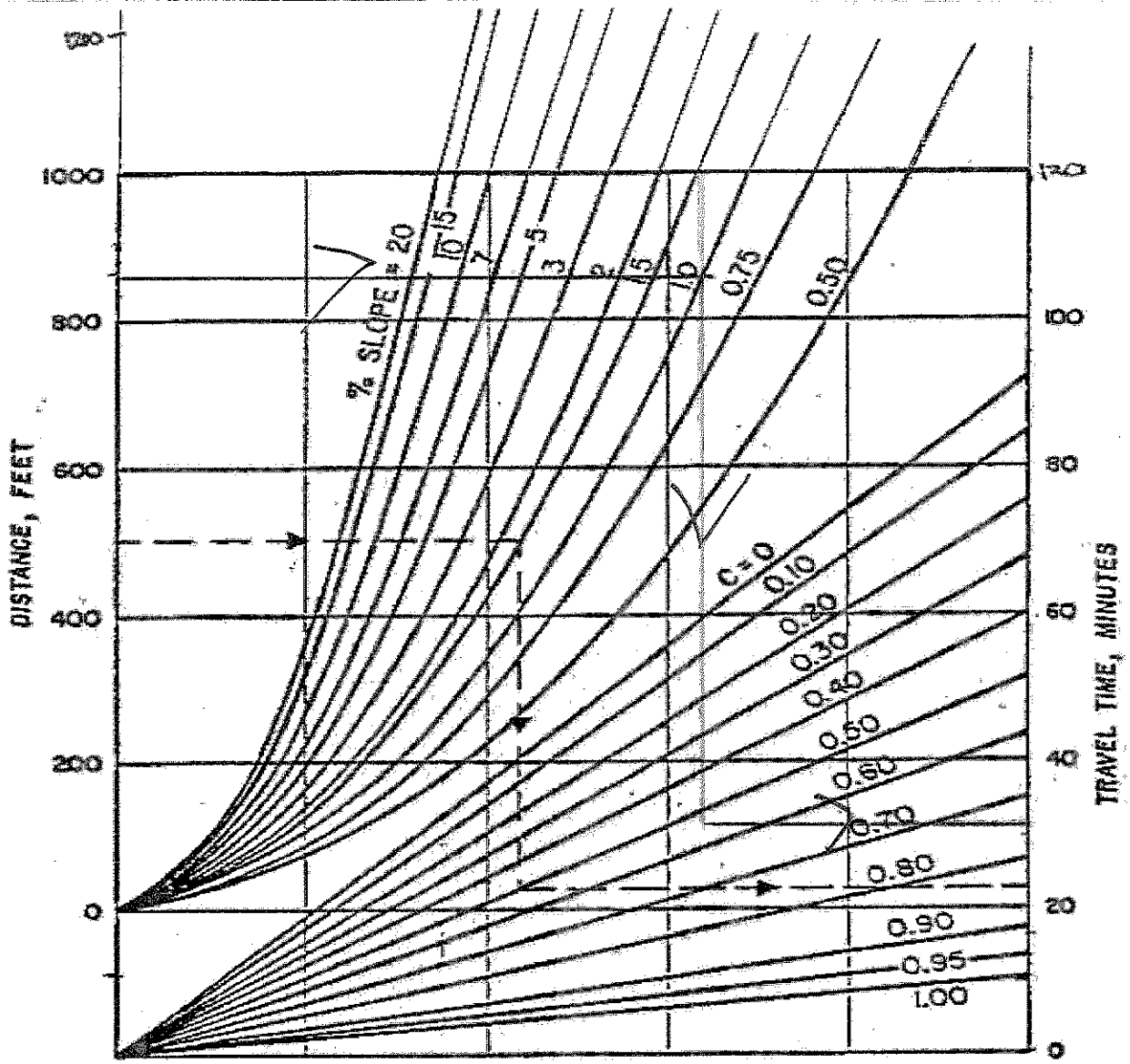
C = 0.5

Slope = 1%

Travel Time = 32 (Tc)

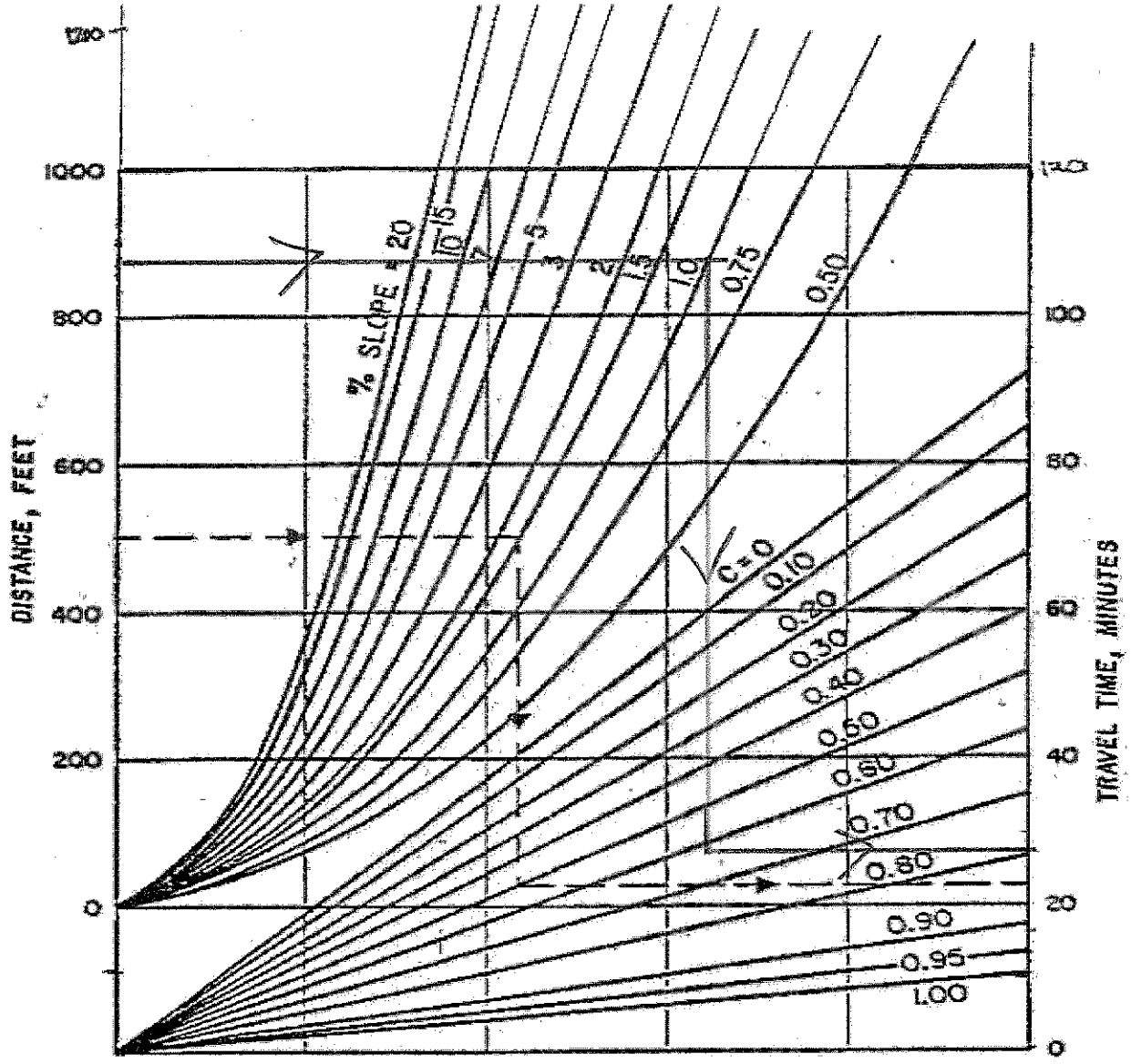
Rain fall Intensity 1.29

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



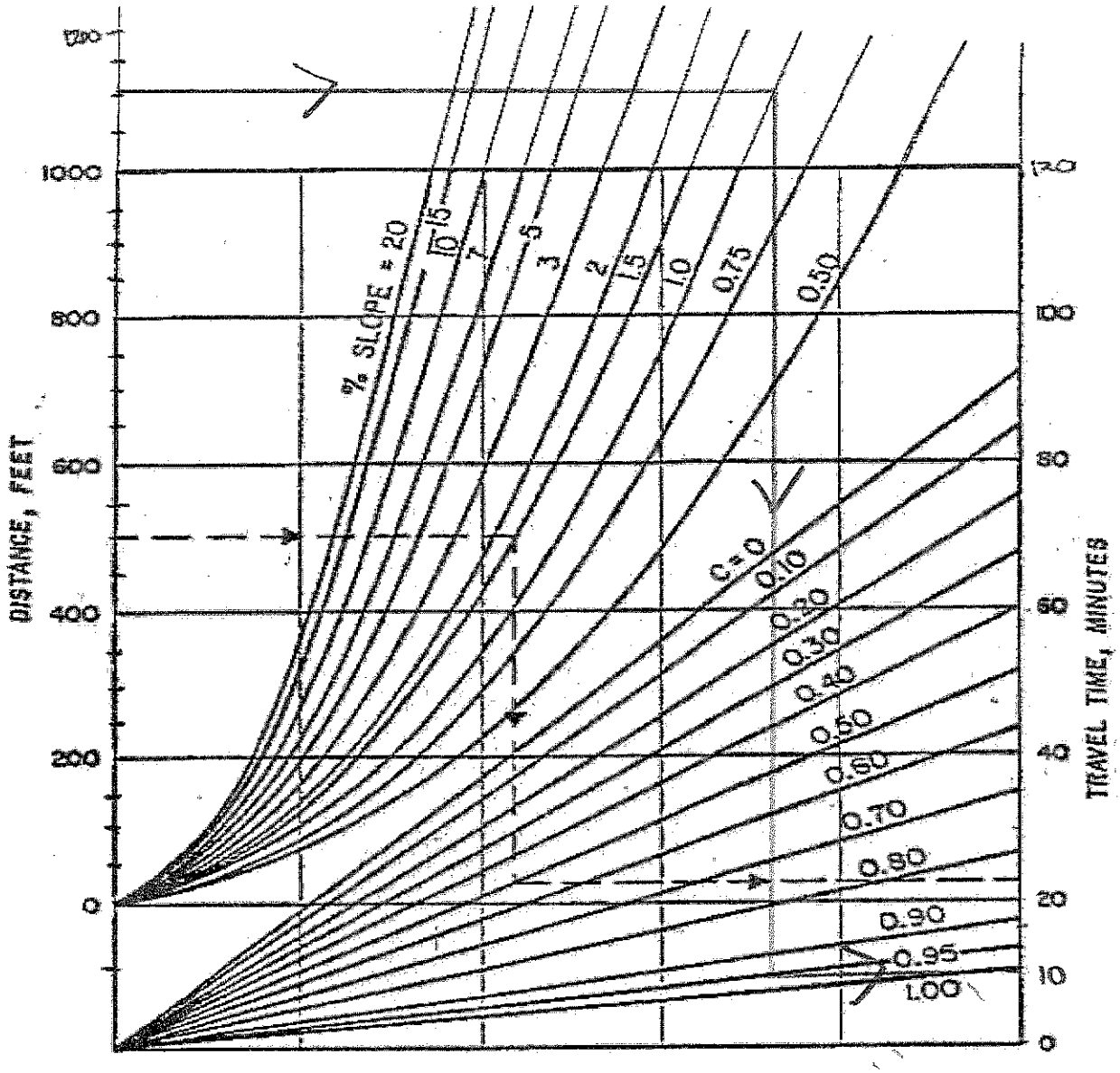
A1 (25-yr.)
Length = 865 ft.
C = 0.535
Slope ≤ 1%
Tc = 31
Rainfall Intensity = 1.52

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A1 (100 yr.)
C = 6.25
Slope ≤ 1%
Length = 865 ft.
Tc = 27
Rainfall Intensity = 2.11

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A2 (13 yr.)

Length = 1091 ft.

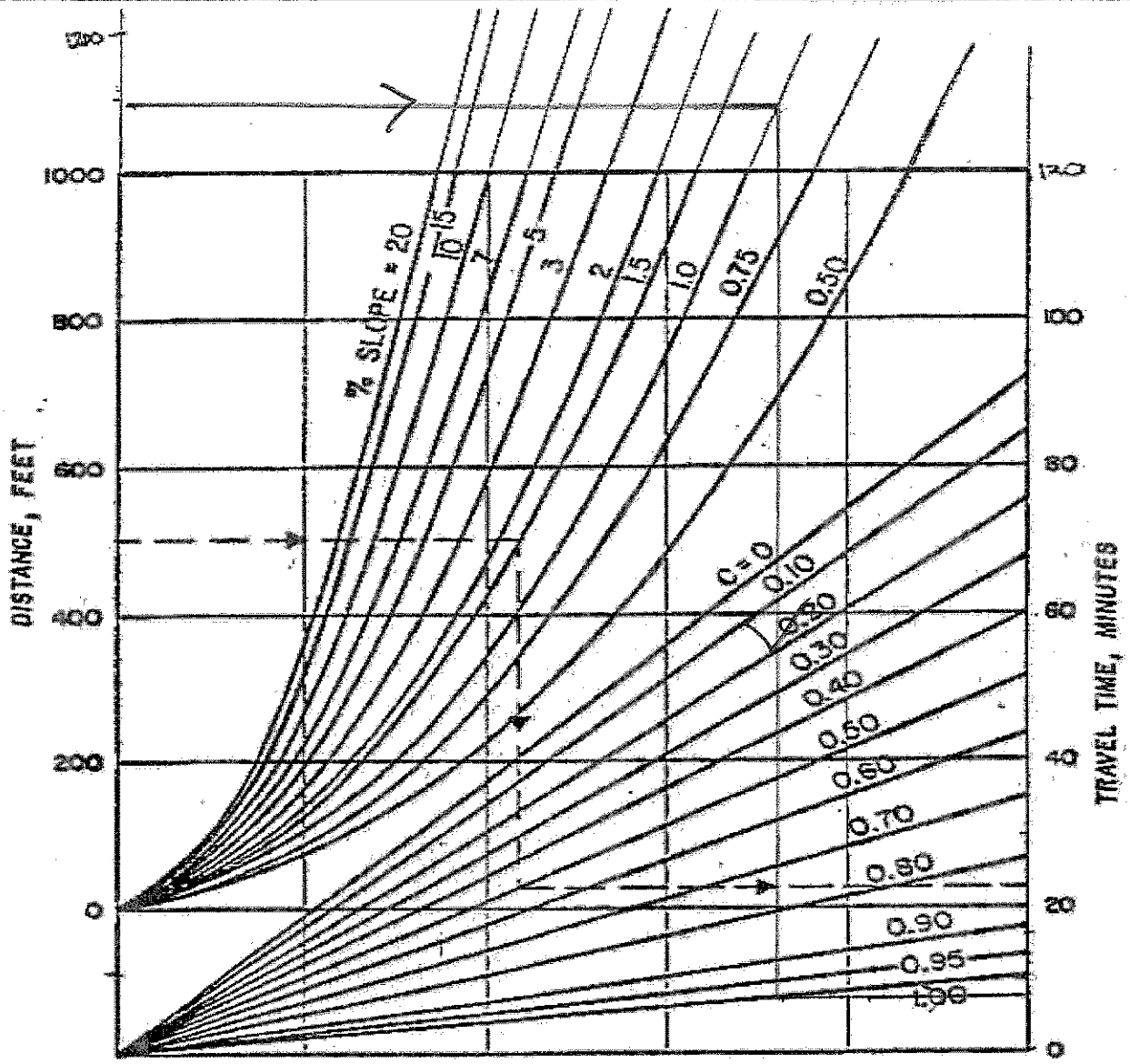
C = 0.95

Slope = 1 %

Travel Time (T_c) = 10

Rainfall Intensity = 2.526

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A2 (25 yr.)

Length = 1091

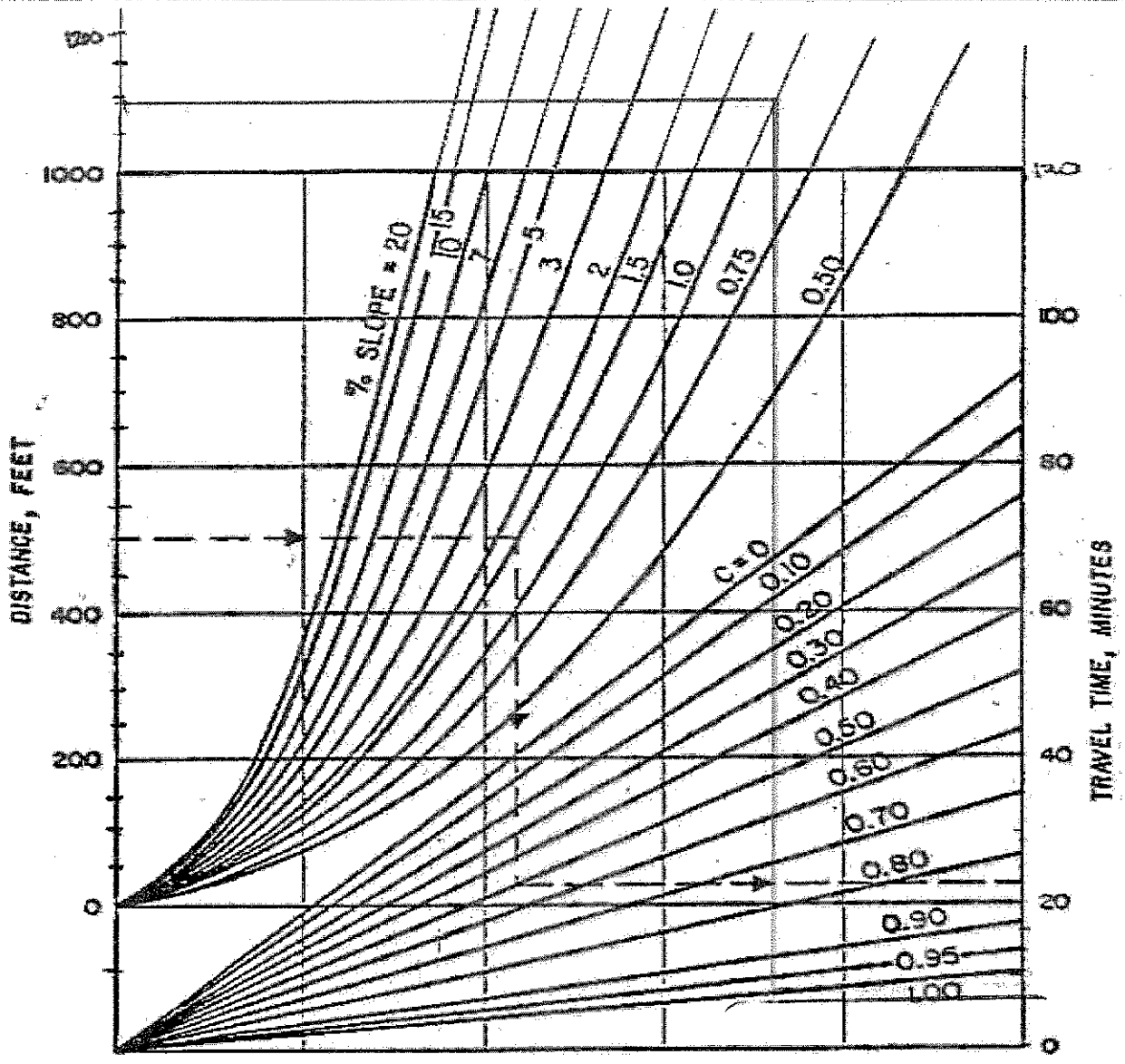
C = 1.0

Slope ≤ 1%

T_c = 7

Rainfall Intensity = 2.19

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A2 (100 yr)

Length = 1091 ft.

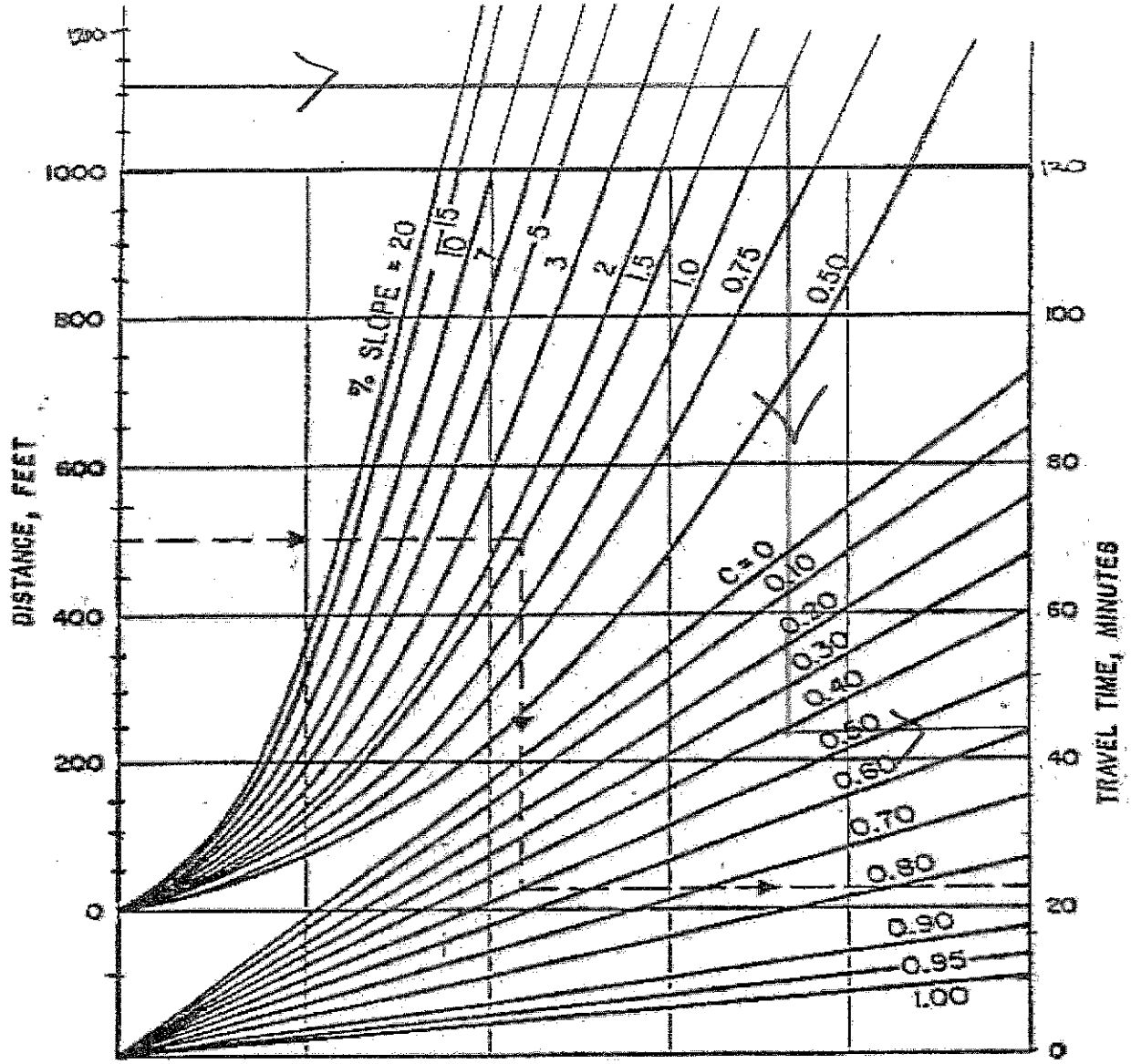
C = 1

Slope ≤ 1 %

Travel Time (T_c) = 6

Rainfall Intensity = 3.41

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (Existing) - 15yr

Length = 1097 ft.

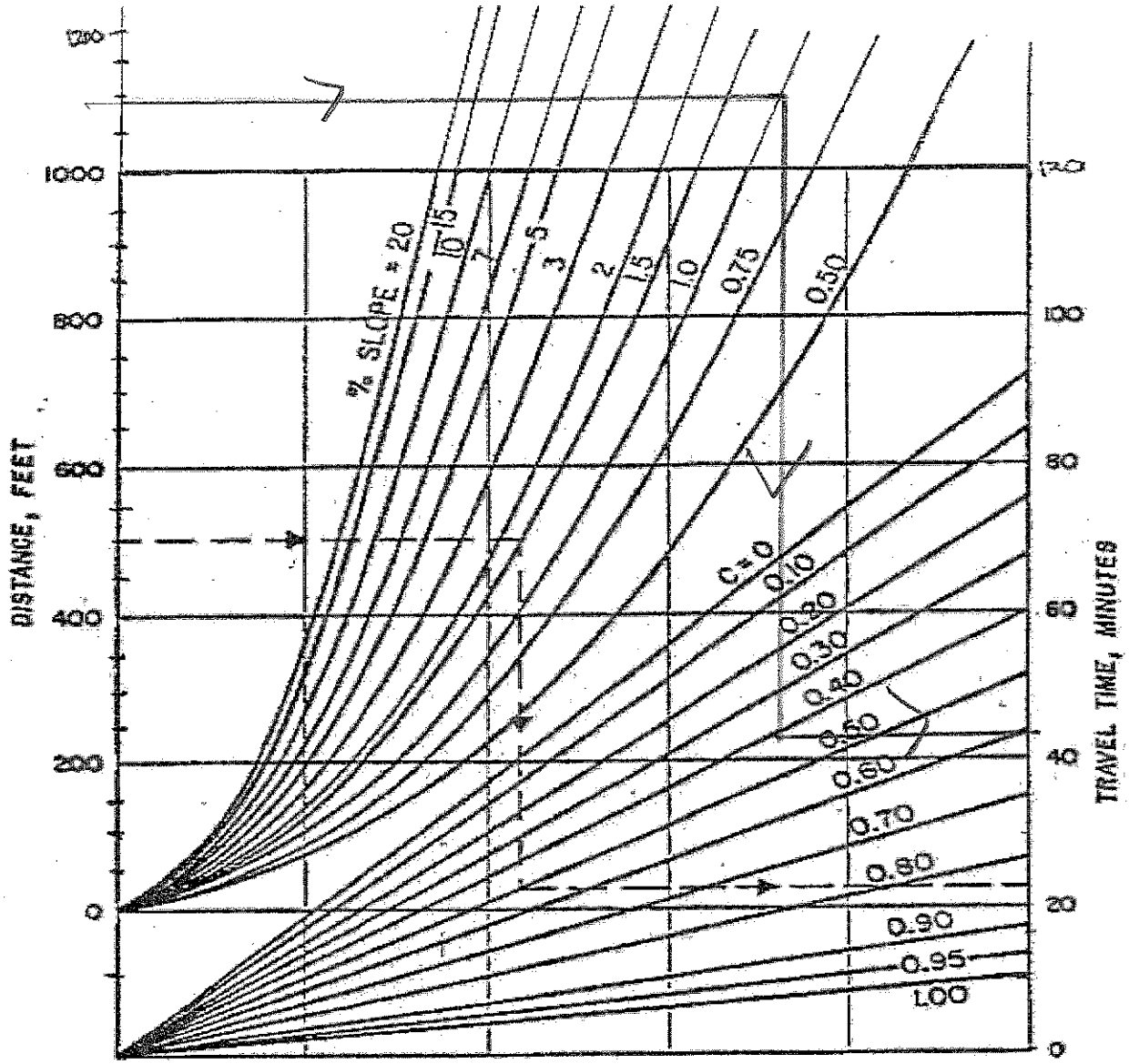
C = 0.4

Slope = ≤ 1%

Travel Time (Tc) = 4 hr

Rainfall Intensity = 11.03

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (25 yr. - Existing Conditions)

Travel Time (T_c) = 43

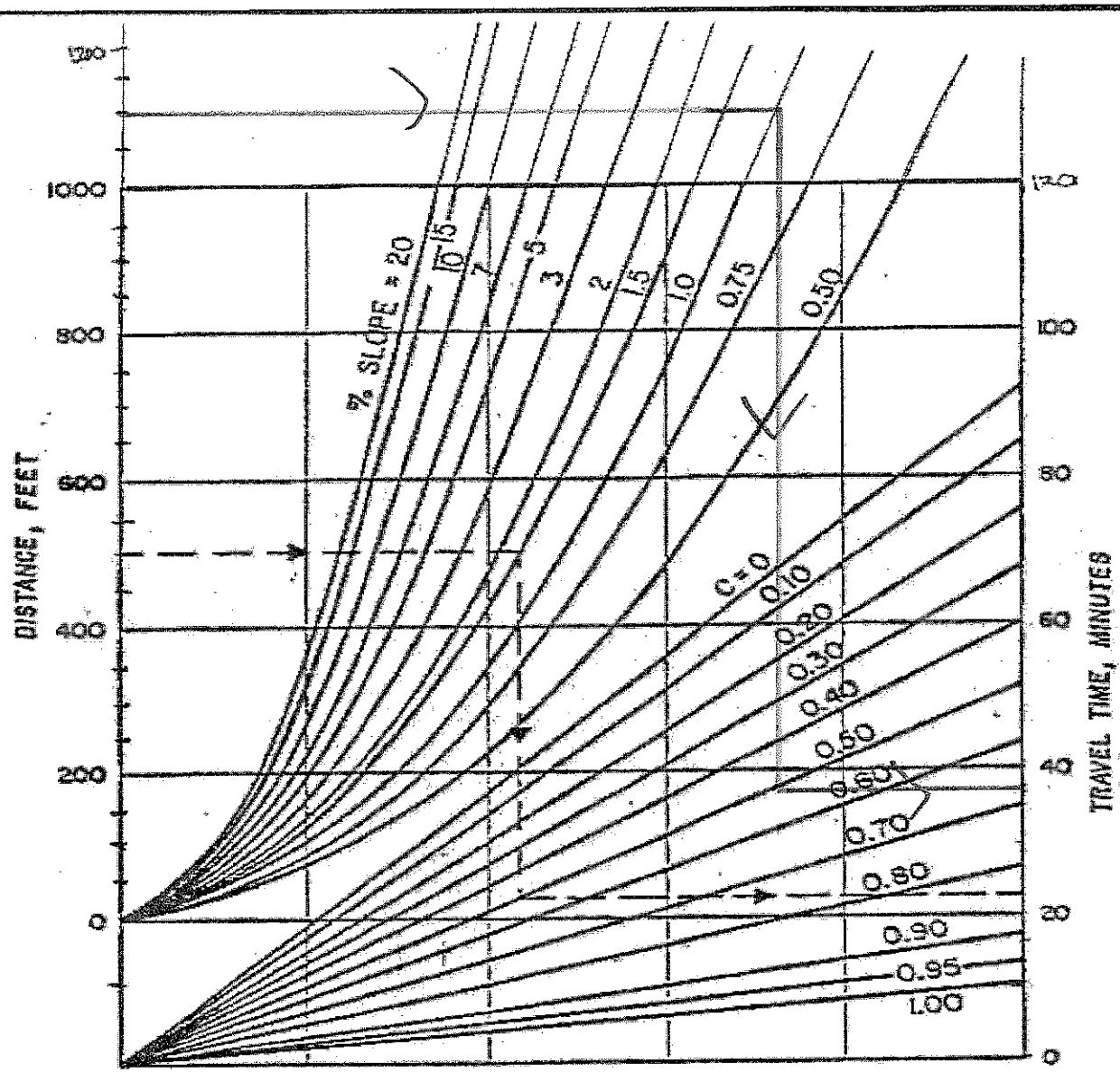
Length = 1097 ft.

C = 0.428

Rainfall Intensity = 1.627

Slope ≤ 1 %

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (100 yr.- Existing Conditions)

Length = 1097 ft.

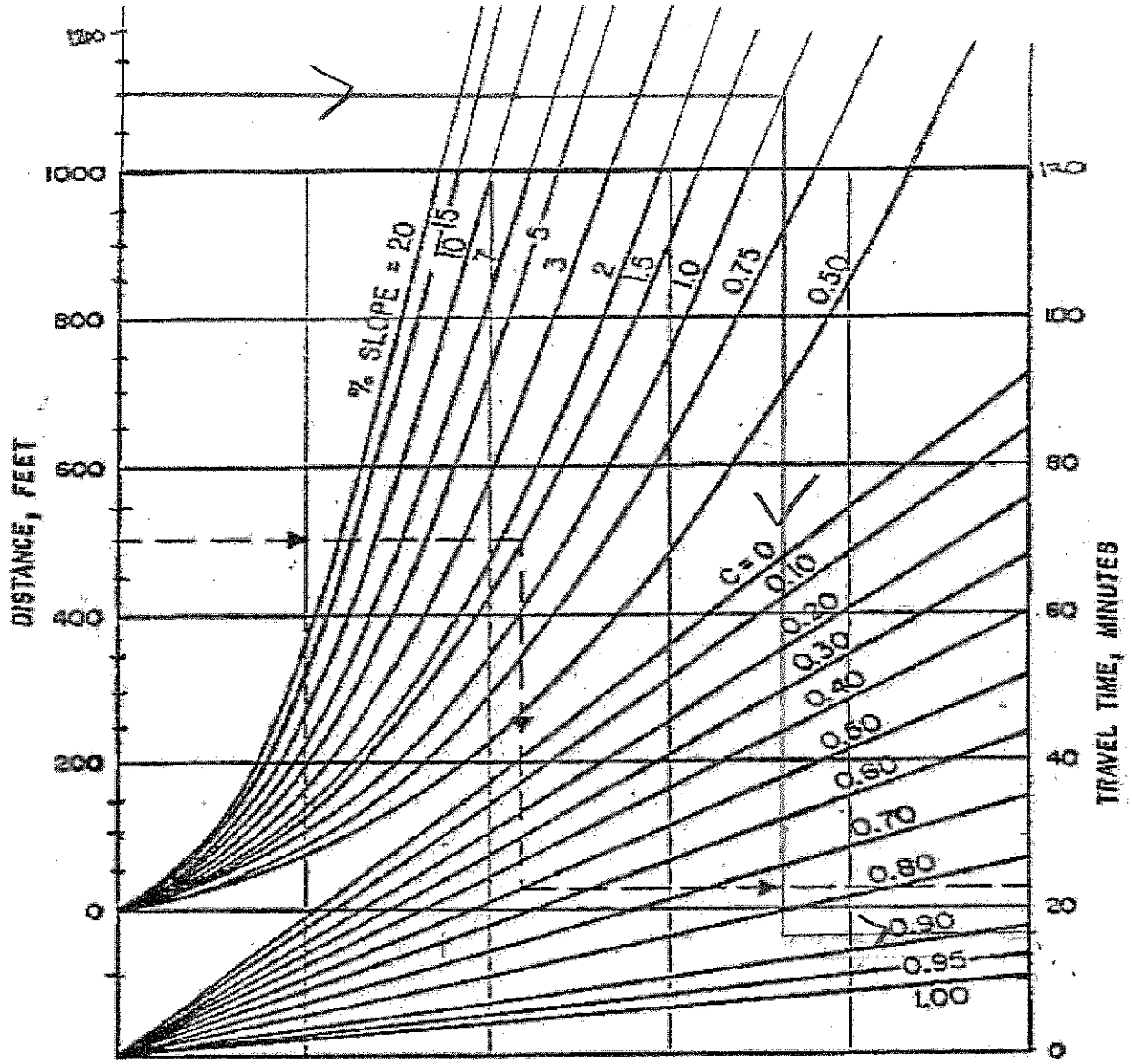
C = 0.5

Slope = ≤1%

Travel Time (T_c) = 37

Rainfall Intensity = 1.80

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (w/ Project) 13 yr.

Length = 1097 ft.

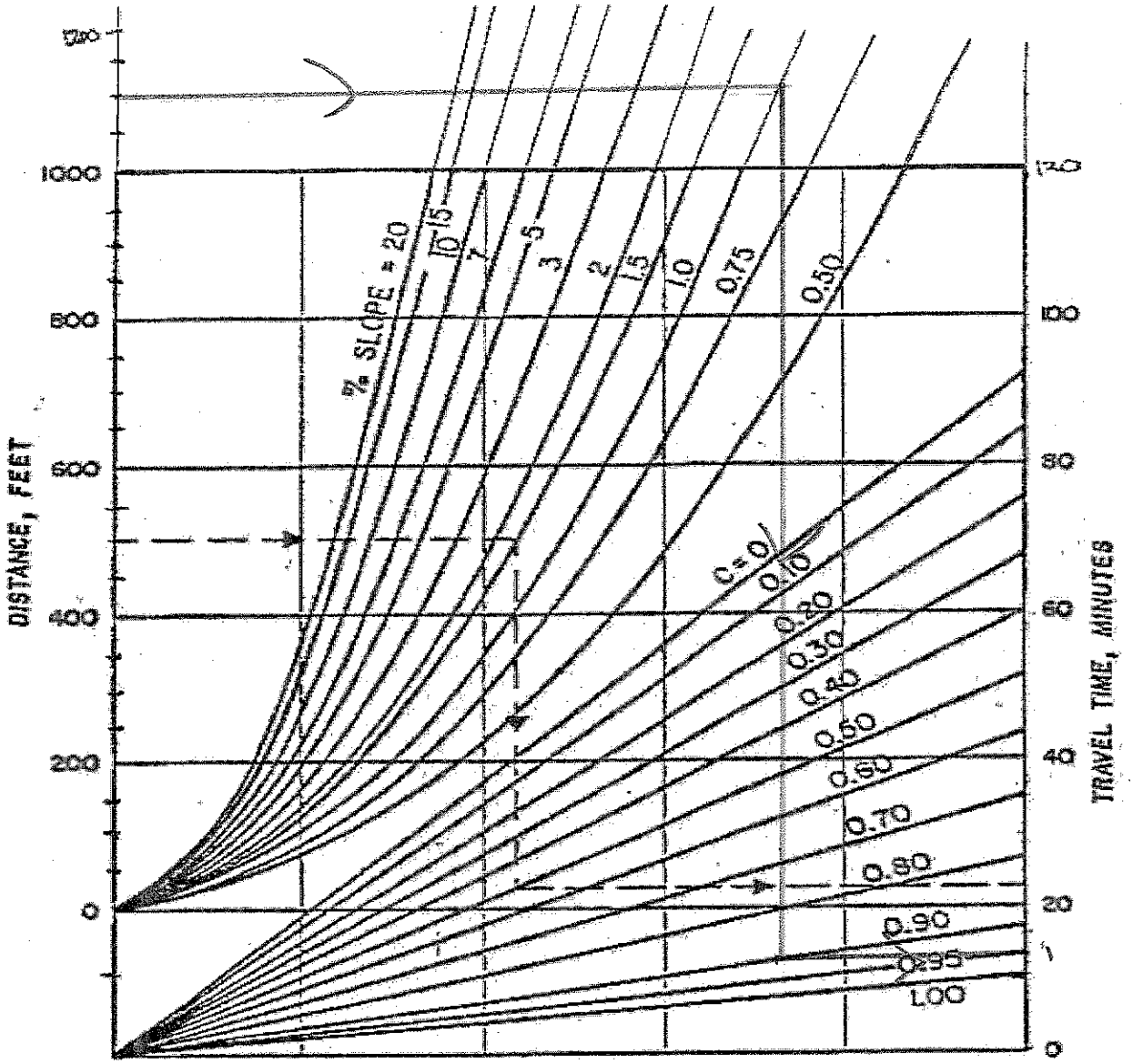
C = 0.83

Slope = < 1 %

Travel Time (T_c) = 16

Rainfall Intensity = 1.78

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (25 yr w/ Project

Length = 1097 ft.

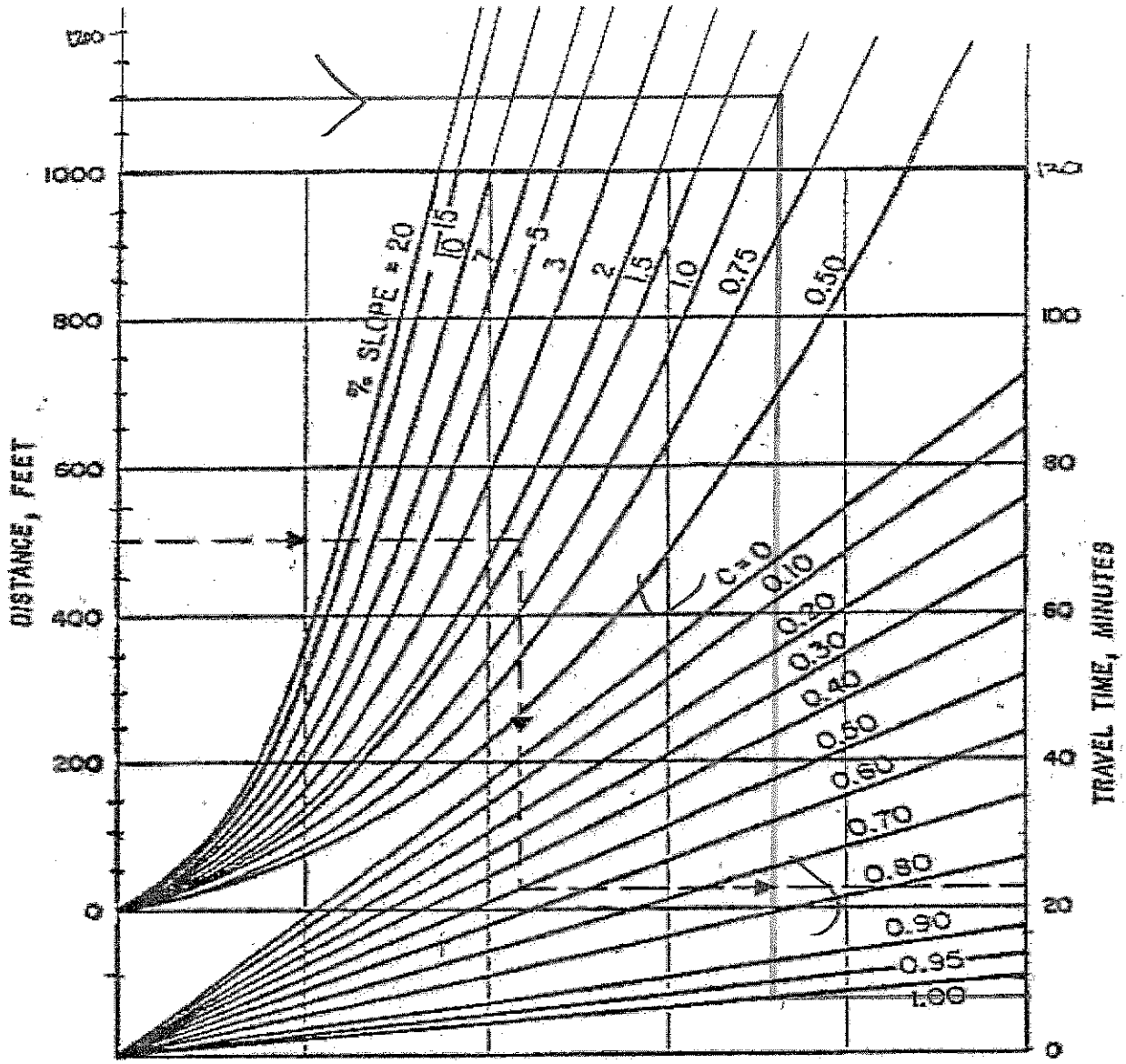
C = 0.91

Slope ≤ 1%

Travel Time (Tc) = 13

Rainfall Intensity = 2.40

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



A3 (100 yr - w/ Project)

Length = 1097 ft.

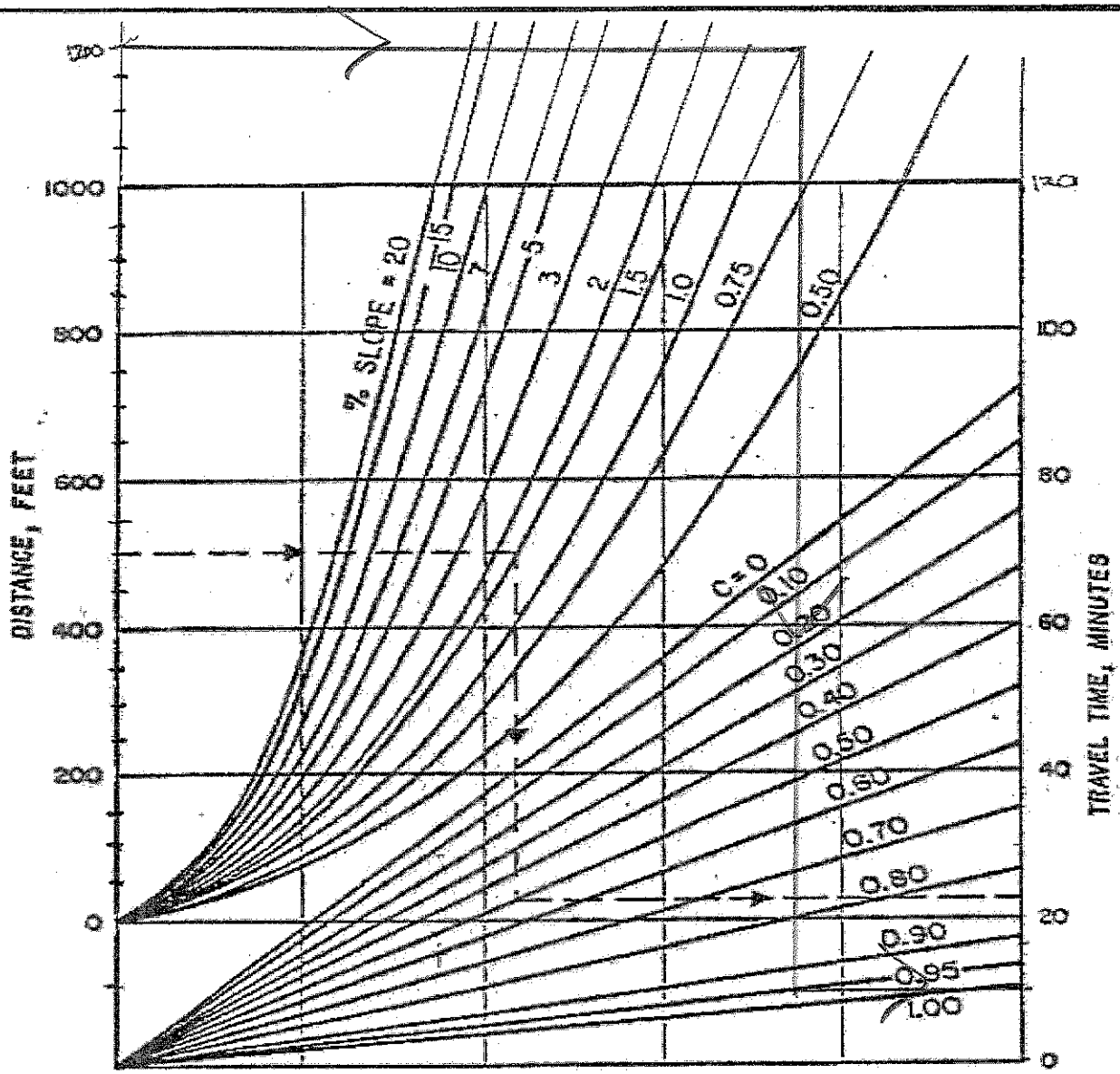
C = 1.0

Slope ≤ 1 %

Travel Time (T_c) = 7

Rainfall Intensity = 4.18

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



B (15yr.)

Length = 1224 ft.

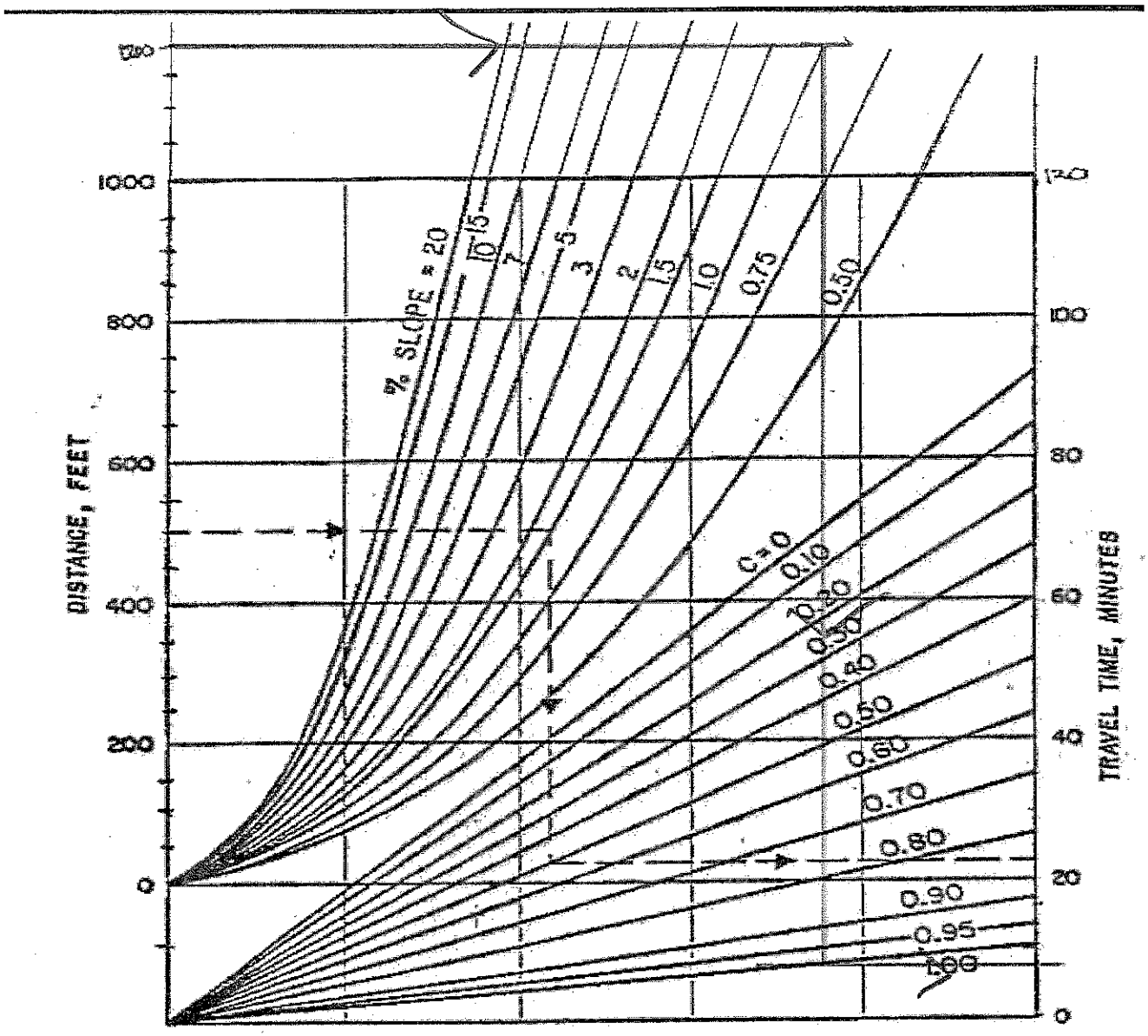
C = 0.95

Slope = ≤ 1%

Travel Time (Tc) = 10

Rainfall Intensity = 2.26

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



B (25 yr)

Length = 1224 ft.

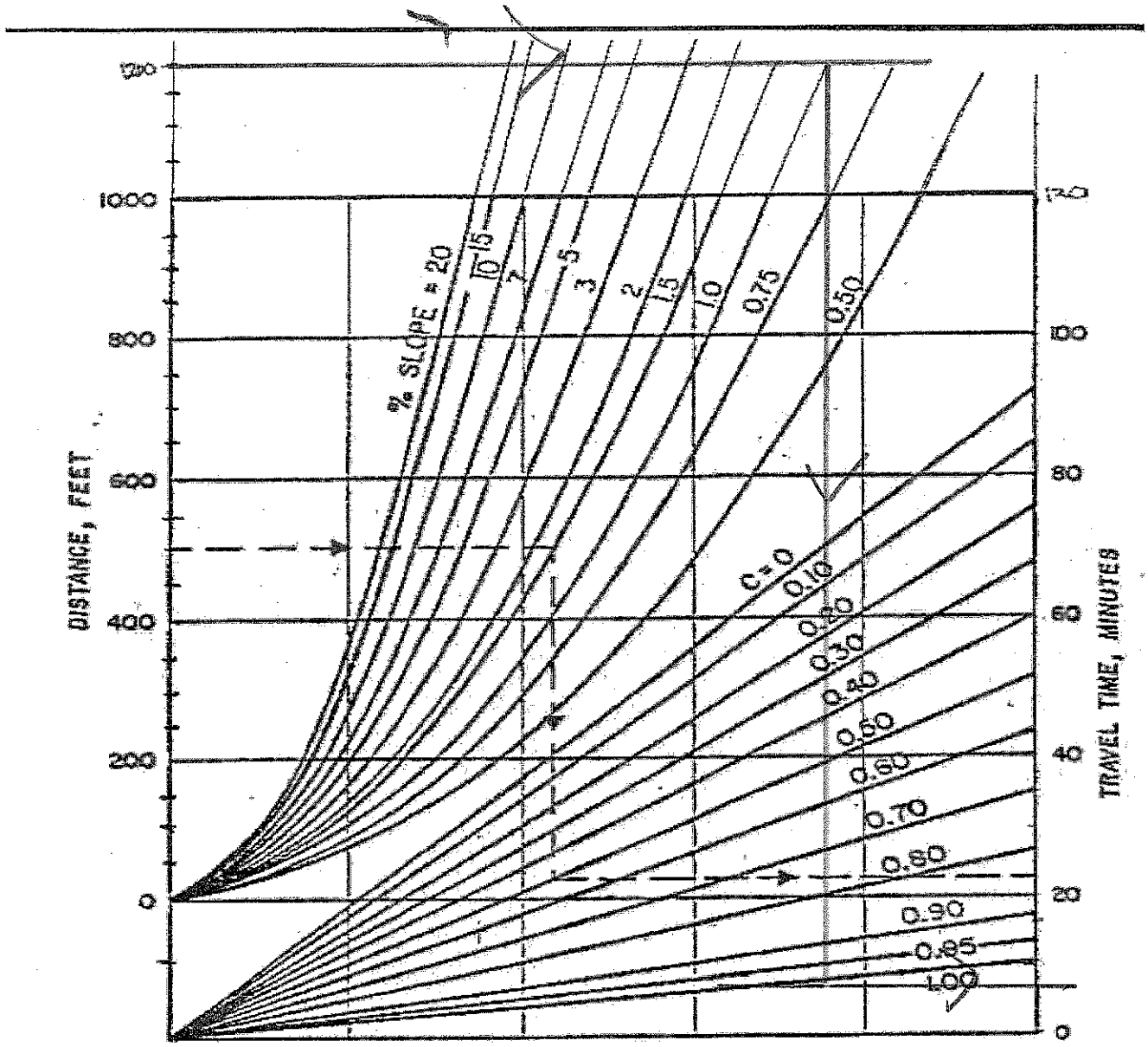
C = 1.0

Slope ≤ 1 %

Travel Time (Tc) = 8

Rainfall Intensity = 2.00

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



B (100yr)

Length = 1224 ft. ~ 1200

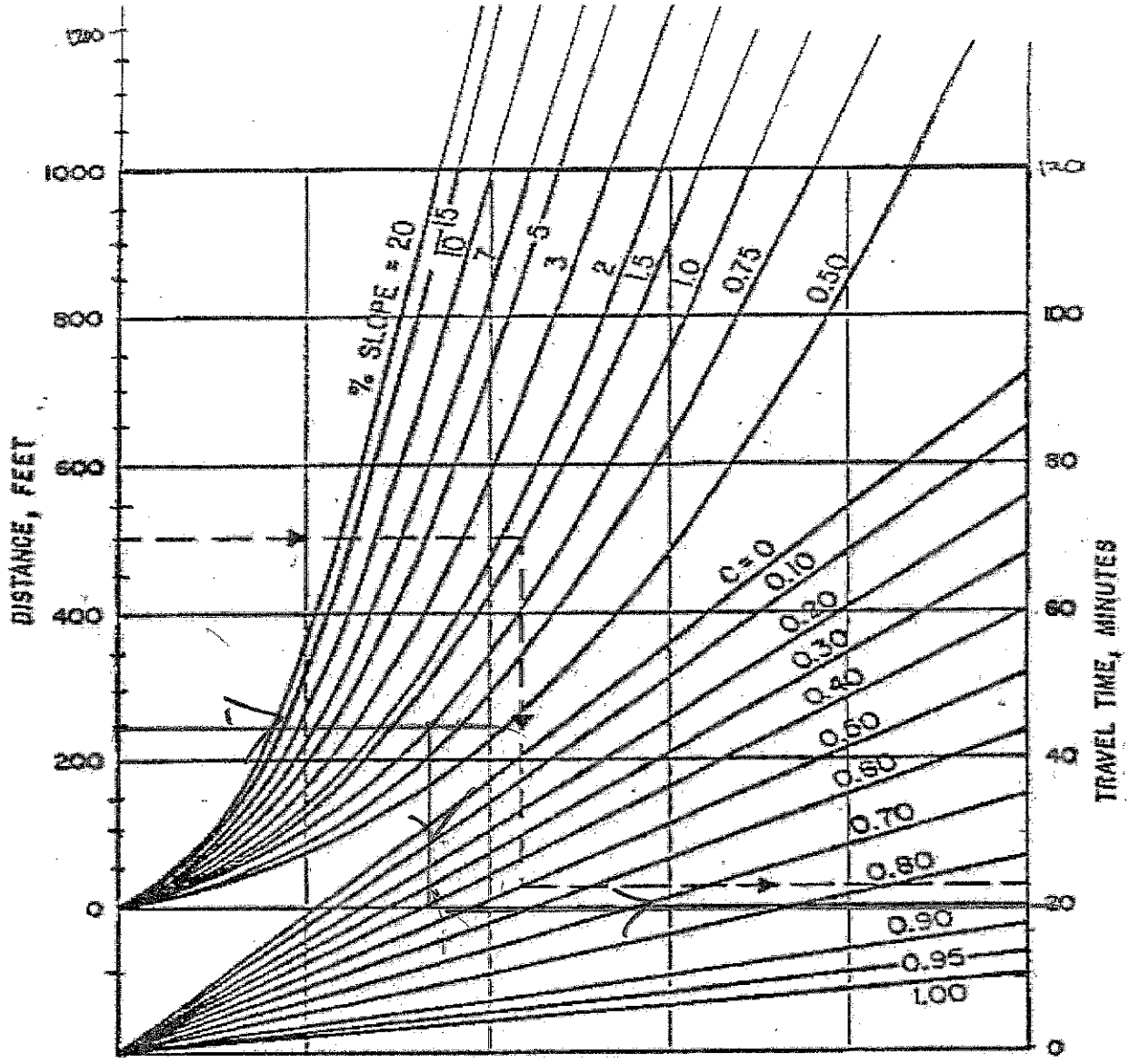
C = 1.0

Slope = ≤ 1%

Travel Time (T_c) = 6

Rainfall Intensity = 4.48

FIGURE 1
TRAVEL TIME FOR OVERLAND FLOW



C 1 (yr.)

Length = 246 ft.

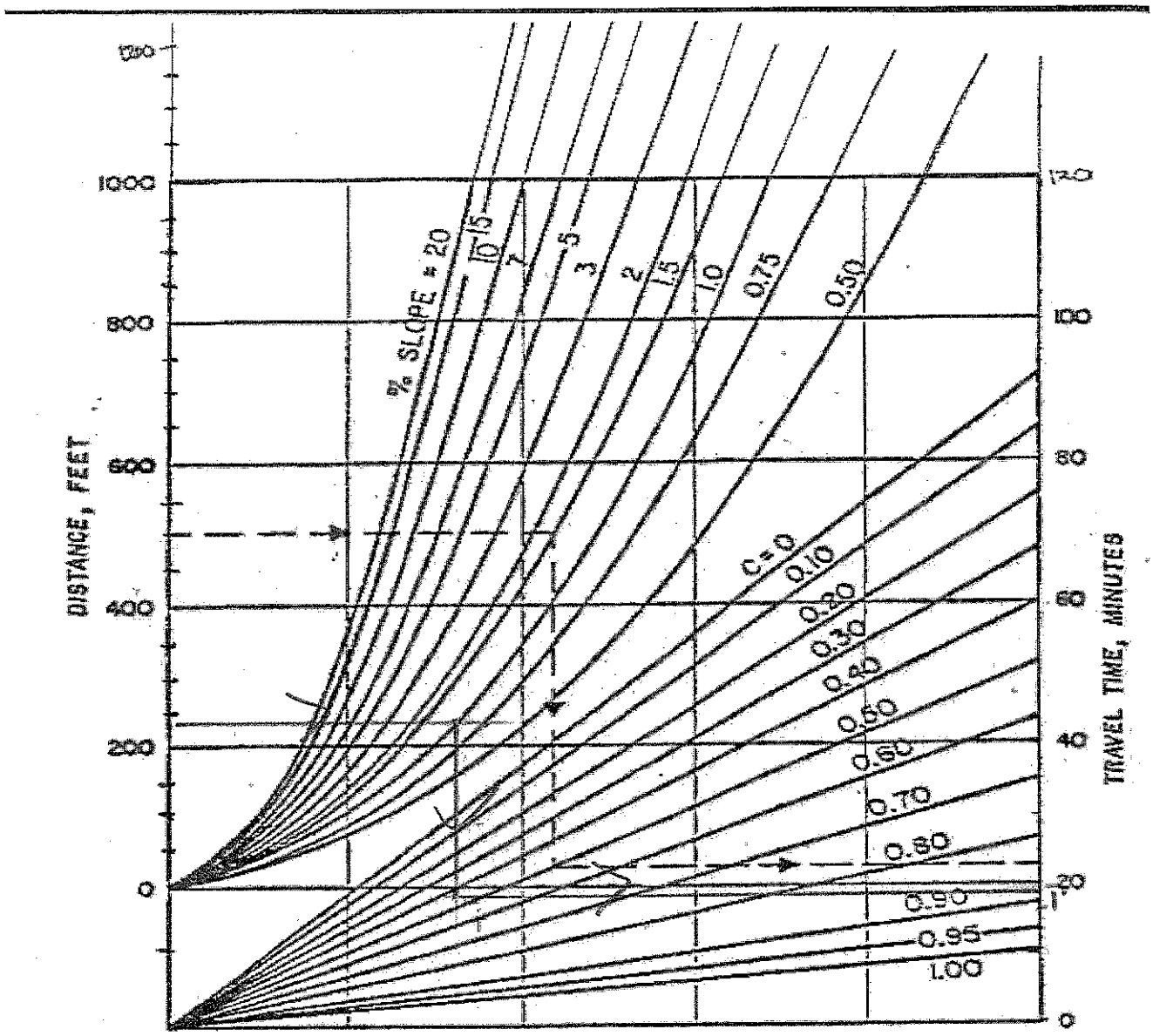
C = 0.4

Slope = ≤ 1 %

Travel Time (Tc) = 20

Rainfall Intensity = 1.57

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



C1 (25yr)

Length = 246 ft.

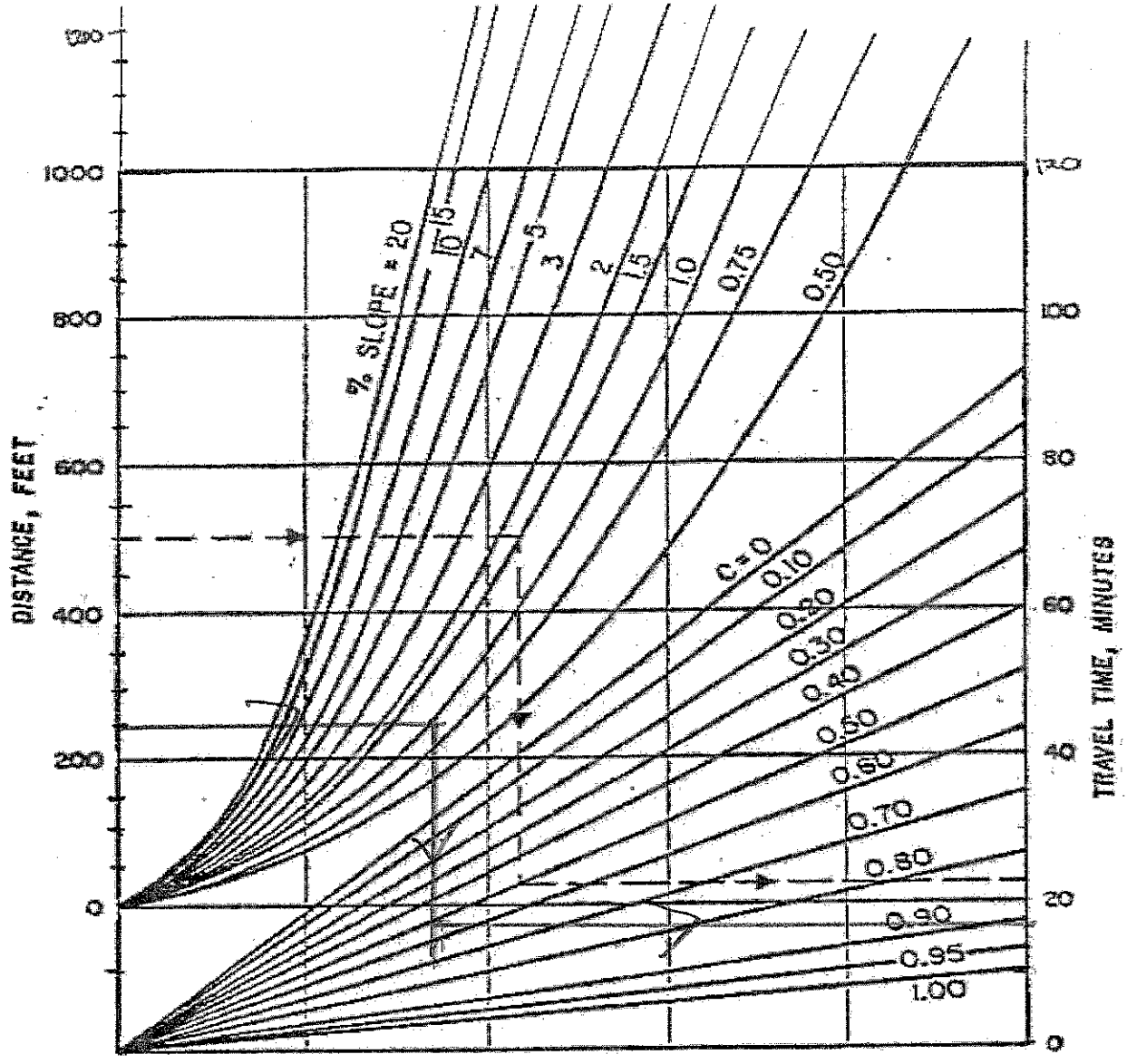
C = 0.428

Slope = ≤ 1%

Travel Time (Tc) = 18

Rainfall Intensity = 2.01

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



C1 (100yr)

Length = 246 ft.

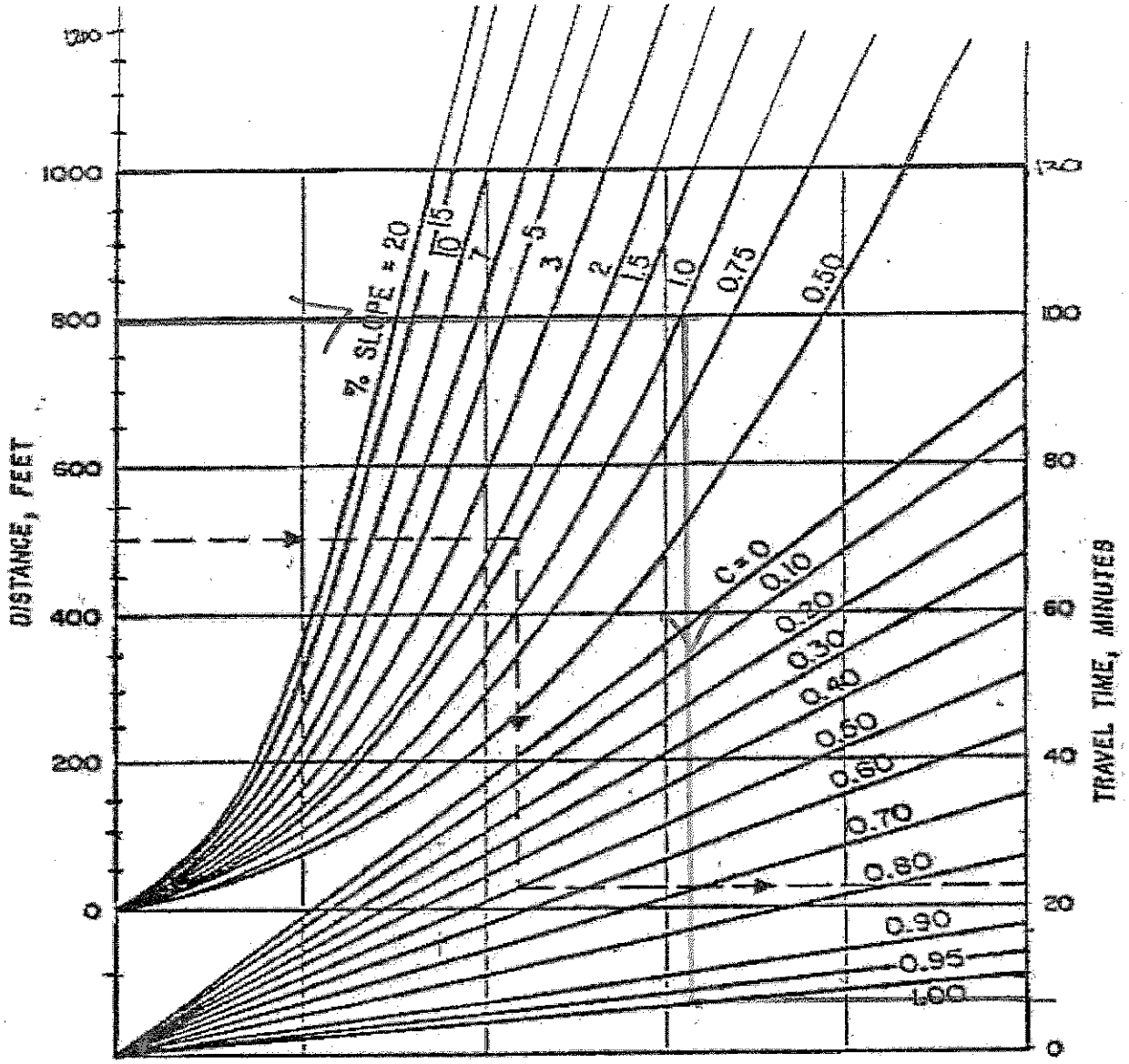
C = 0.5

Slope ≤ 1%

Travel Time (Tc) = 15

Rainfall Intensity = 2.90

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Cla (15 yr.)

Length = 792 ft.

C = 1.0

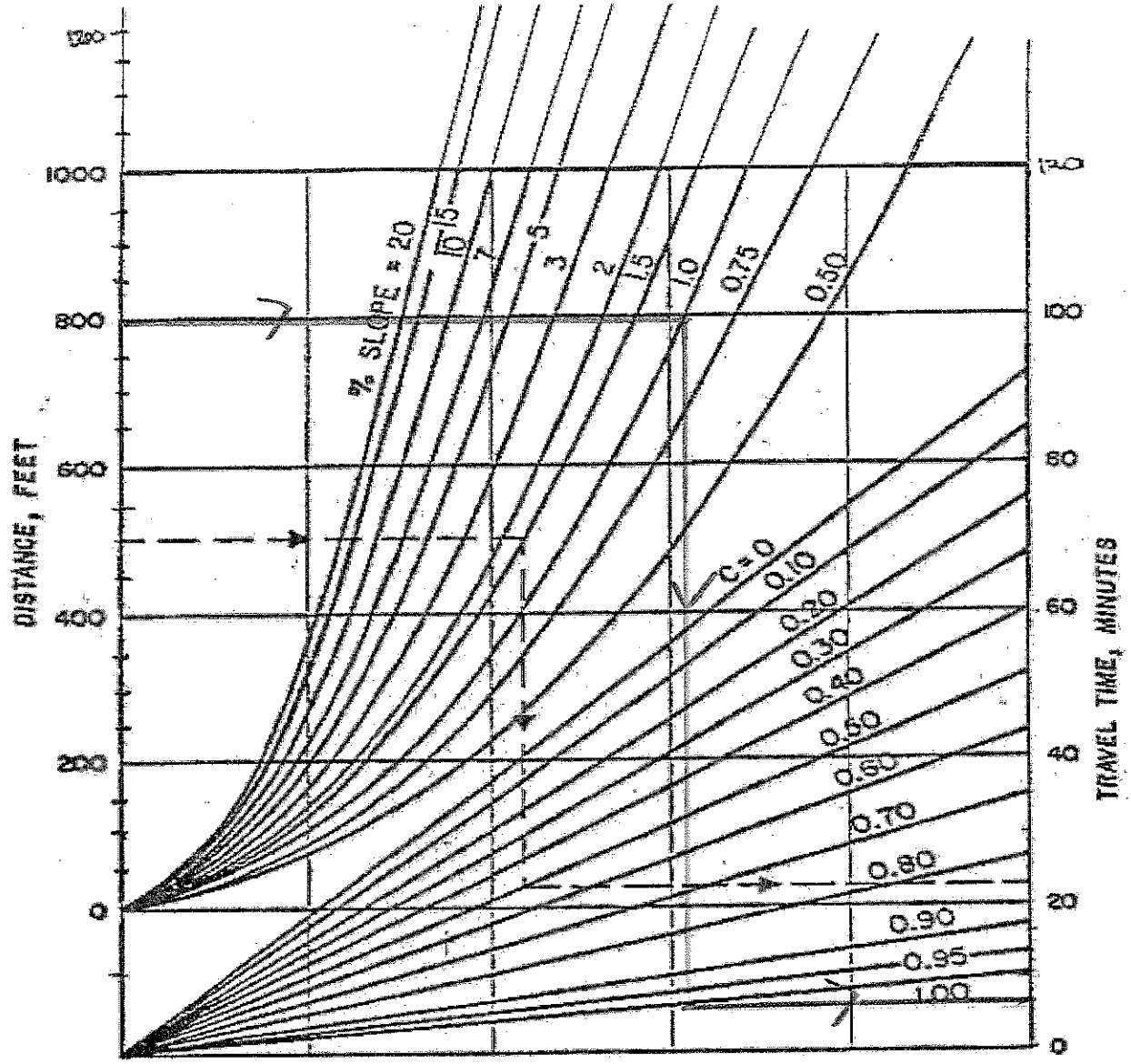
Slope = ≤ 1%

Travel Time (Tc) = 7

Rainfall Intensity = 2.65

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

Note: Calcs apply to Clb



Cla (25yr.)

Length = 792 ft.

C = 1.0

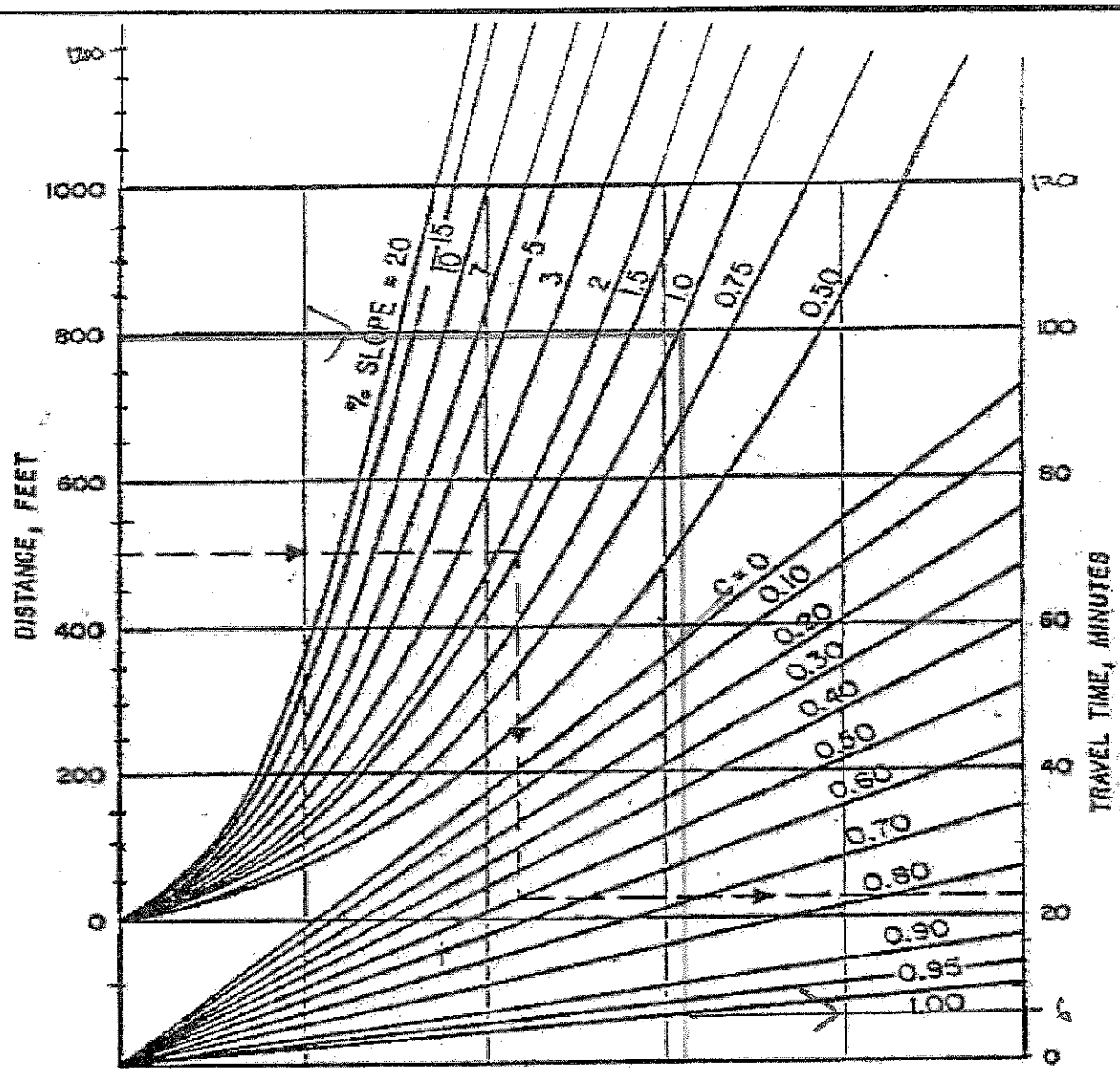
Slope = ≤1%

Travel Time (Tc) = 6

Rainfall Intensity = 3.41

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

Calcs. also apply to C1b



Cla (100 yr)

Length = 792 ft.

C = 1.0

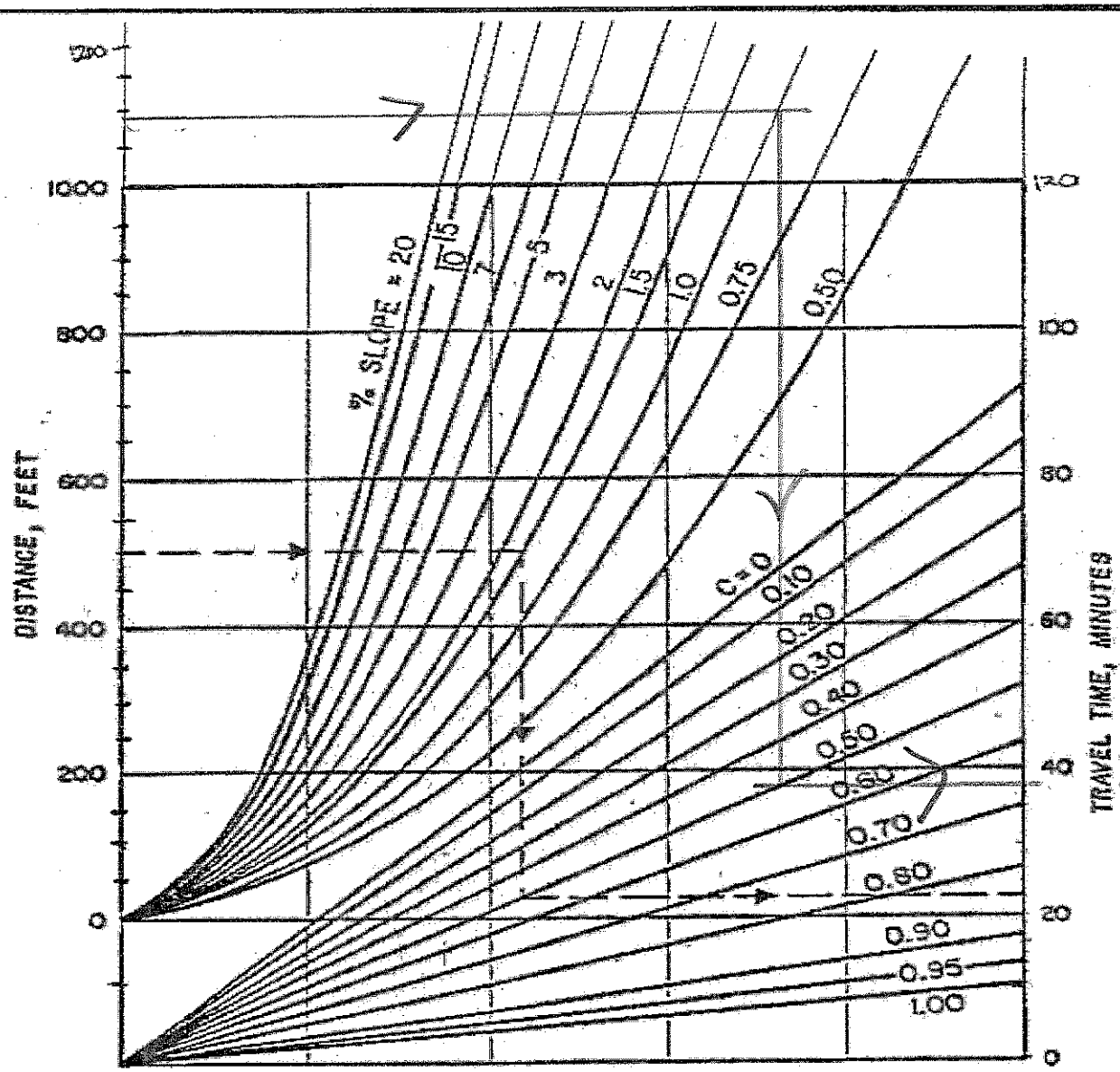
Slope = ≤ 1%

Travel Time (Tc) = 6

Rainfall Intensity = 4.48

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

Note: Calcs. also apply to Clb.



D1 (15 yr.)

Length = 1089 ft.

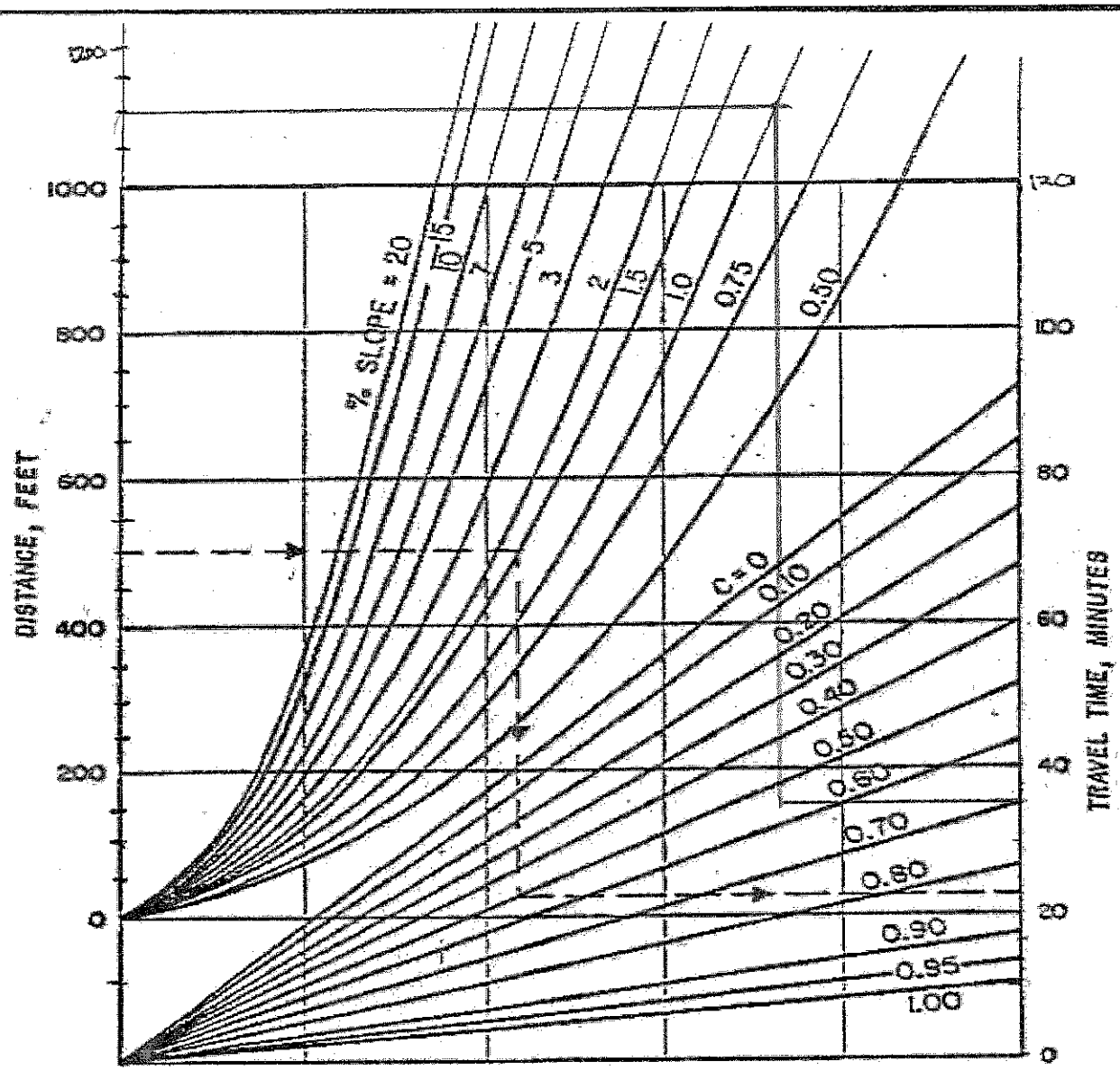
C = 0.5

Slope = ≤1%

Travel Time (Tc) = 37

Rainfall Intensity = 1.14

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D1 (25 yr.)

Length = 1089 ft.

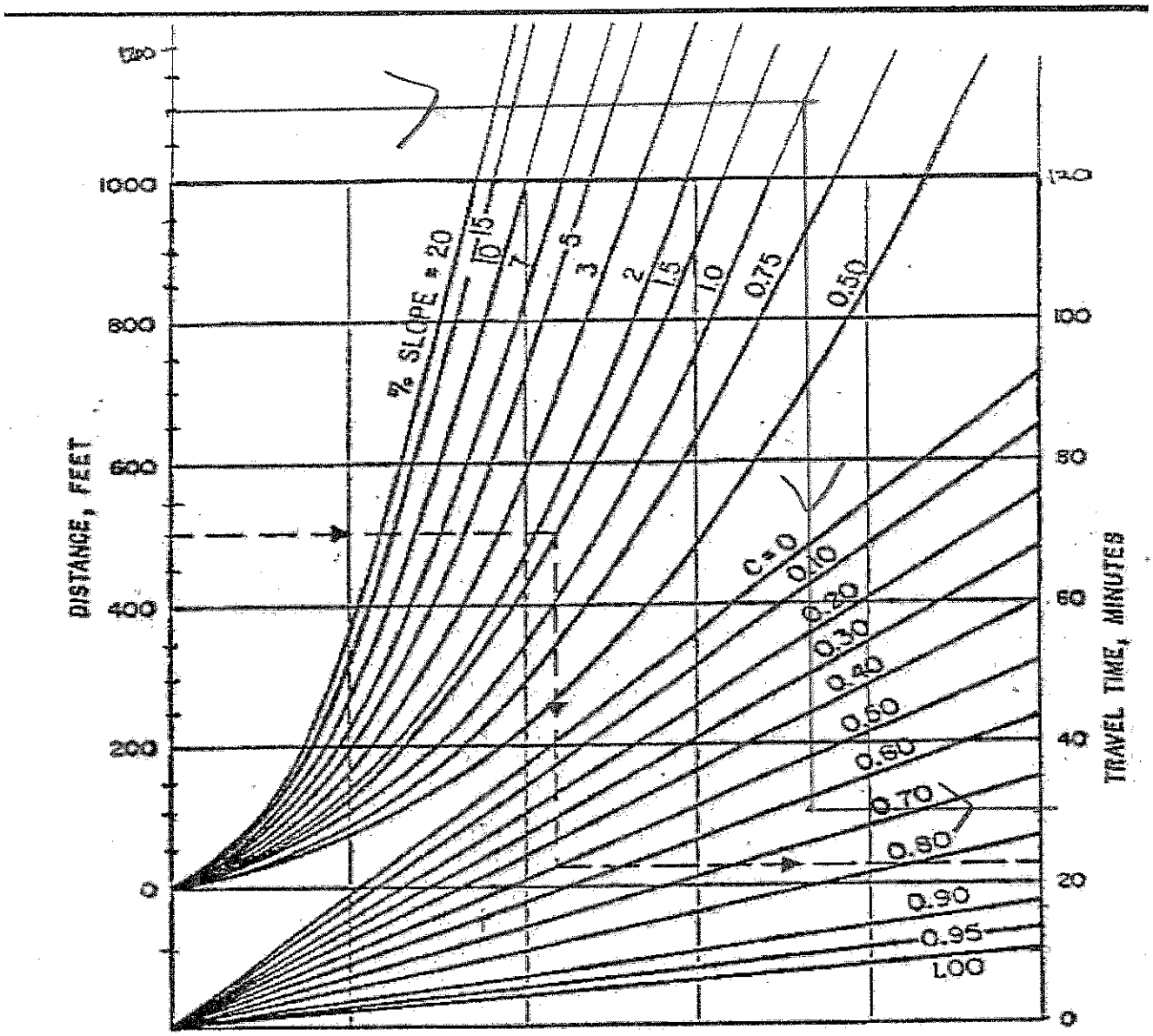
C = 0.54

Slope = ≤ 1%

Travel Time (Tc) = 34

Rainfall Intensity = 1.43

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D1 (100 yr)

Length = 1089 ft.

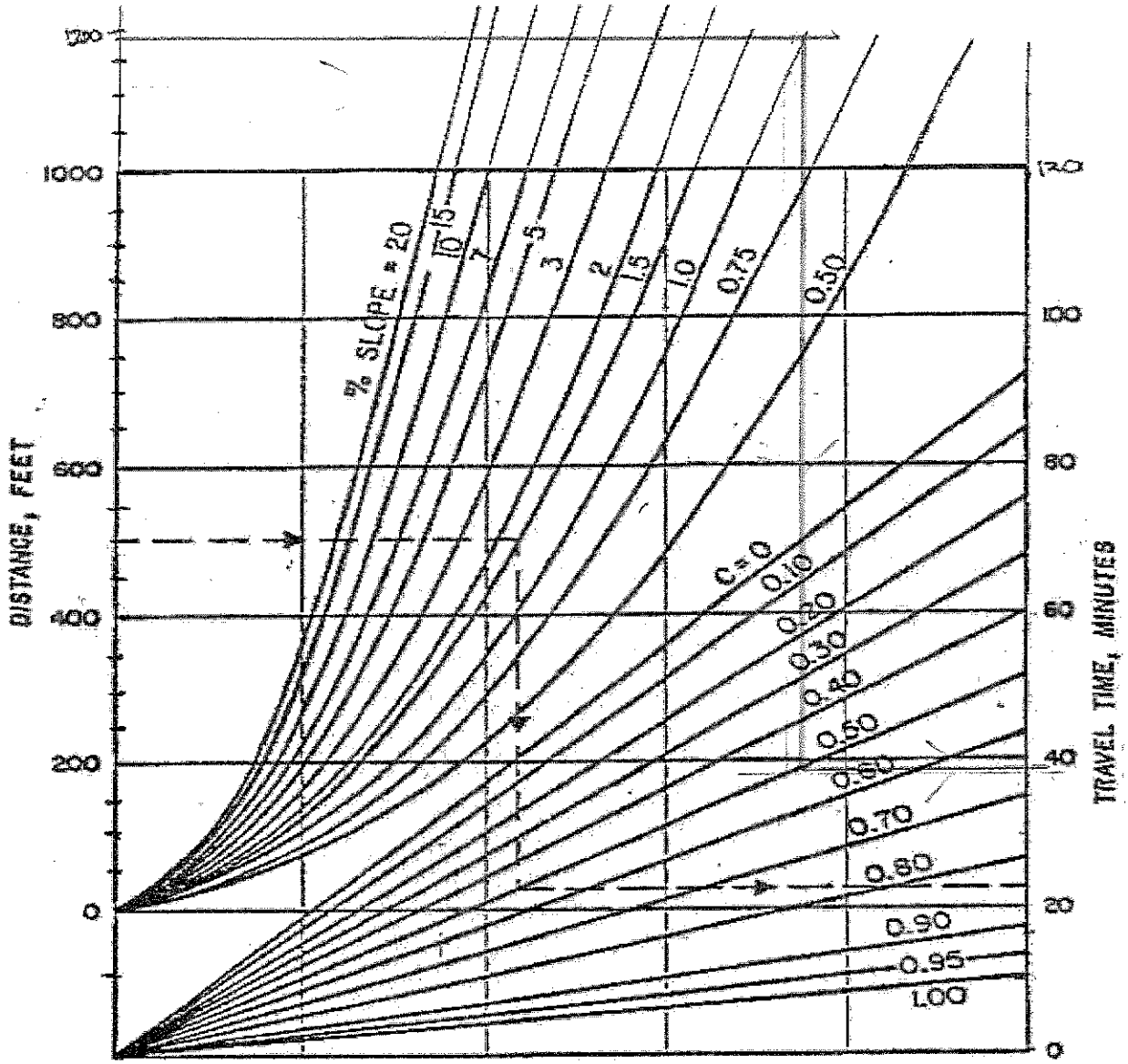
C = 0.63

Slope ≤ 1%

Travel Time (Tc) = 30

Rainfall Intensity = 2.02

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D2 (15 yr.

Length = 1184 ft.

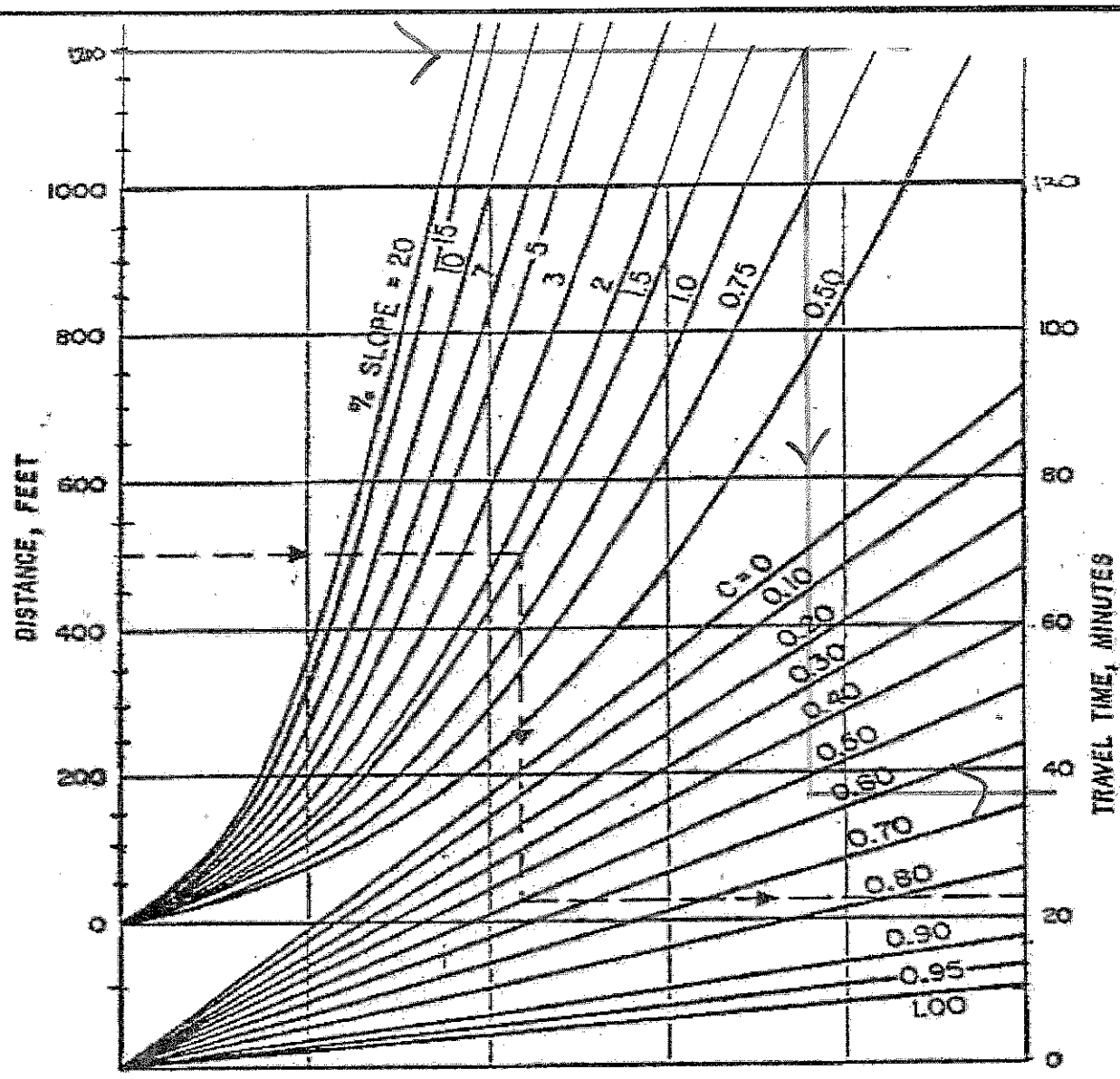
C = 0.5

Slope = ≤ 1 %

Travel Time (T_c) = 38

Rainfall Intensity = 1.12

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D2 (25 yr)

Length = 1184 ft.

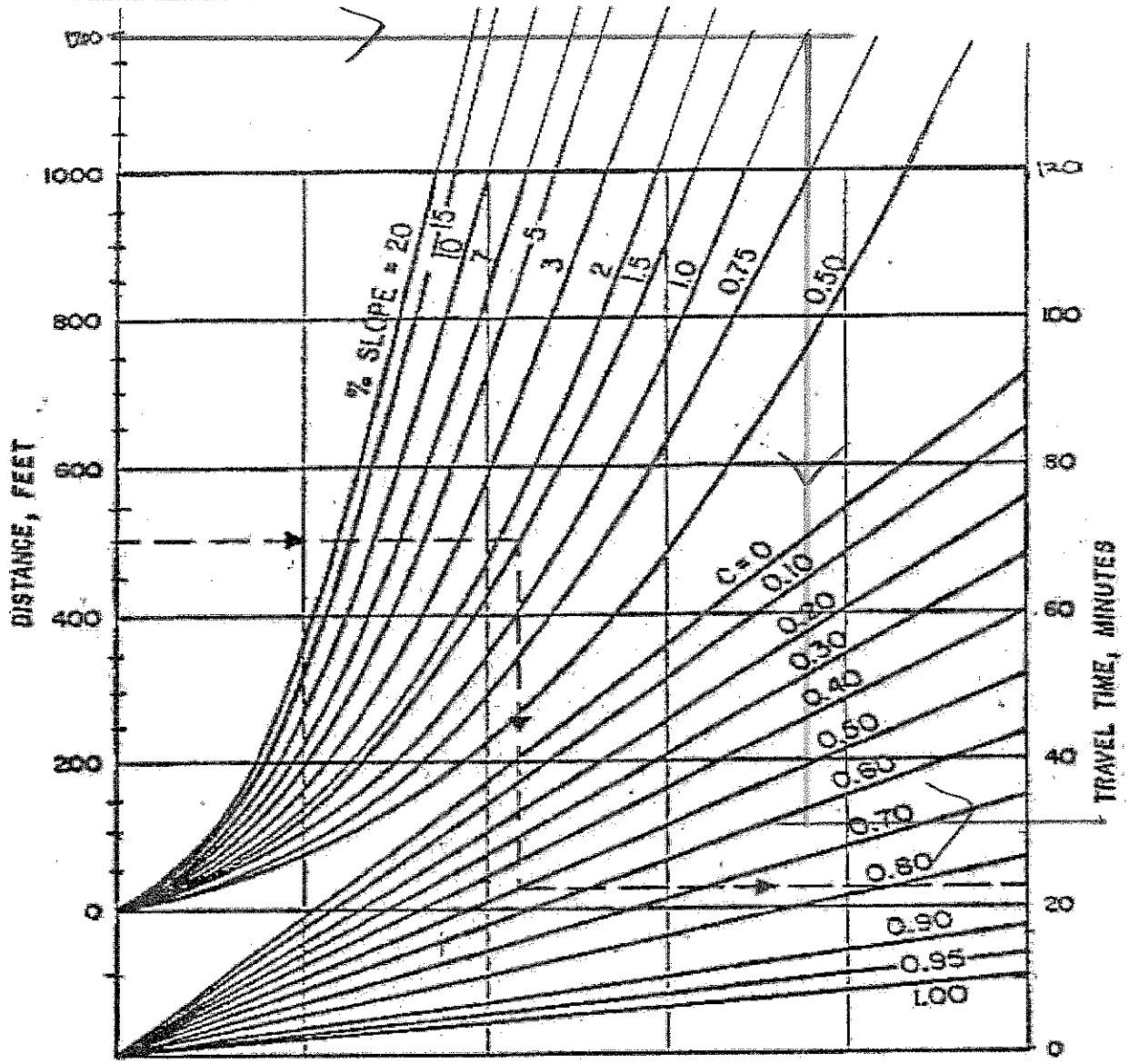
C = 0.54

Slope = ≤ 1%

Travel Time (Tc) = 35

Rainfall Intensity = 1.41 in

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D2 (100 yr)

Length = 1184 ft.

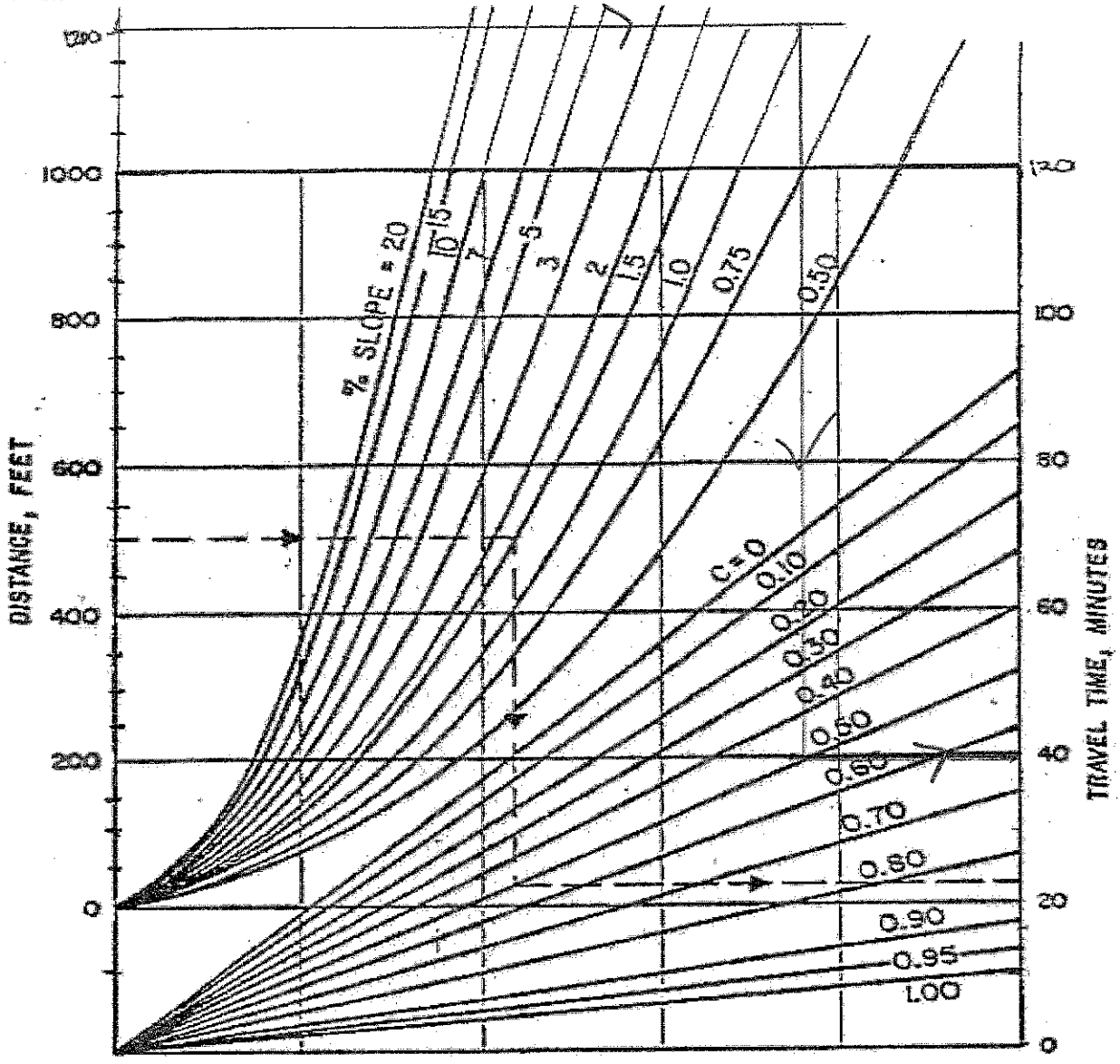
C = 0.63

Slope = 51 %

Travel Time (Tc) = 32

Rainfall Intensity = 1.96

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D3 (15yr.)

Length = 1271 ft. → 1200

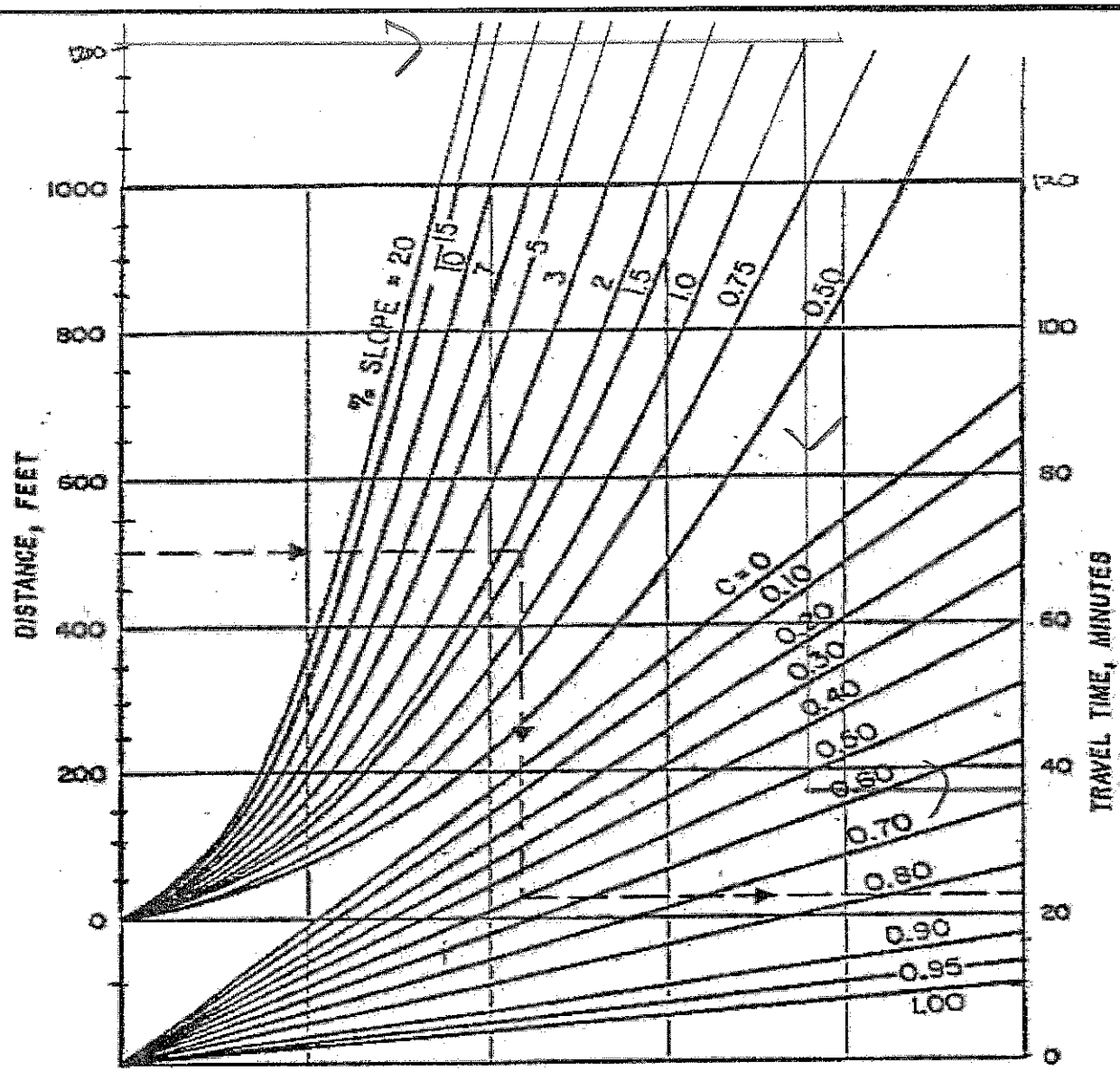
C = 0.5

Slope = ≤ 1%

Travel Time (T_c) = 39

Rainfall Intensity 1.11

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D3 (25yr)

Length = 1271 ft. 1200

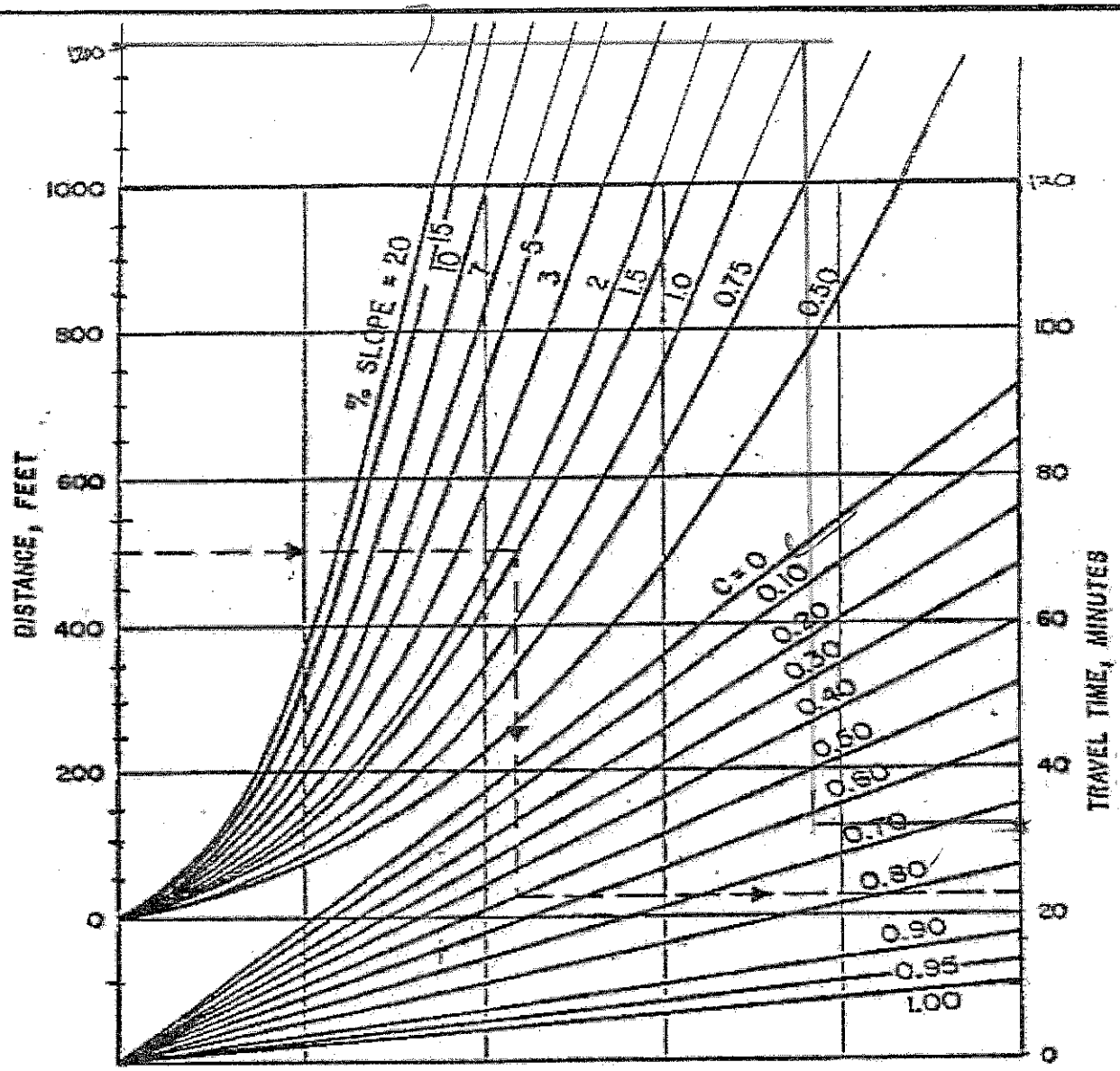
C = 0.54

Slope = ≤ 1%

Travel Time (Tc) = 36

Rainfall Intensity = 2.40

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D3 (100 yr)

Travel Time (Tc) = 32

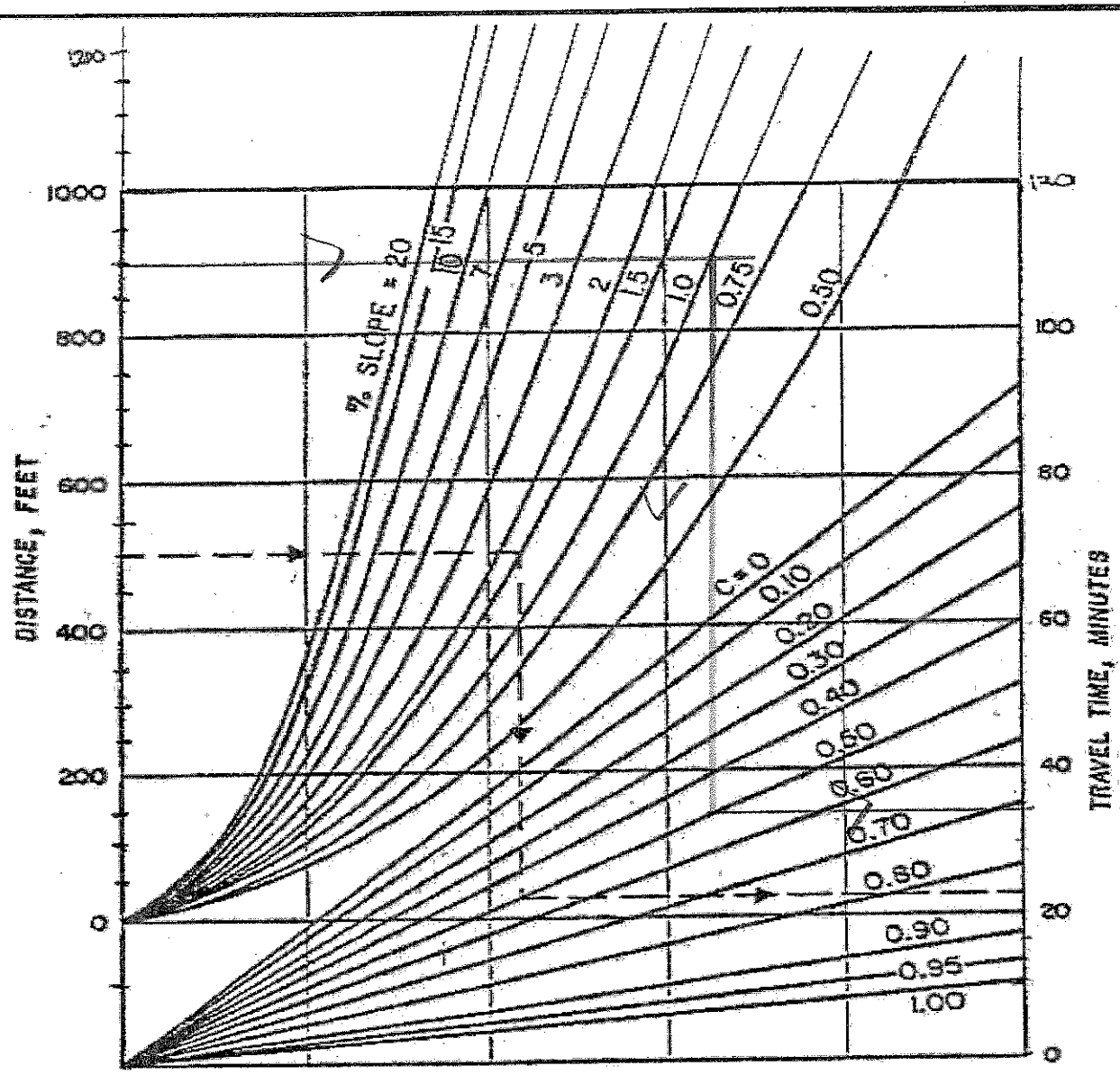
Length = 1271 ft. - 1200

C = 0.63

Slope = ≤ 1%

Rainfall Intensity = 1.96

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D4 (15 yr.)

Length = 893 ft.

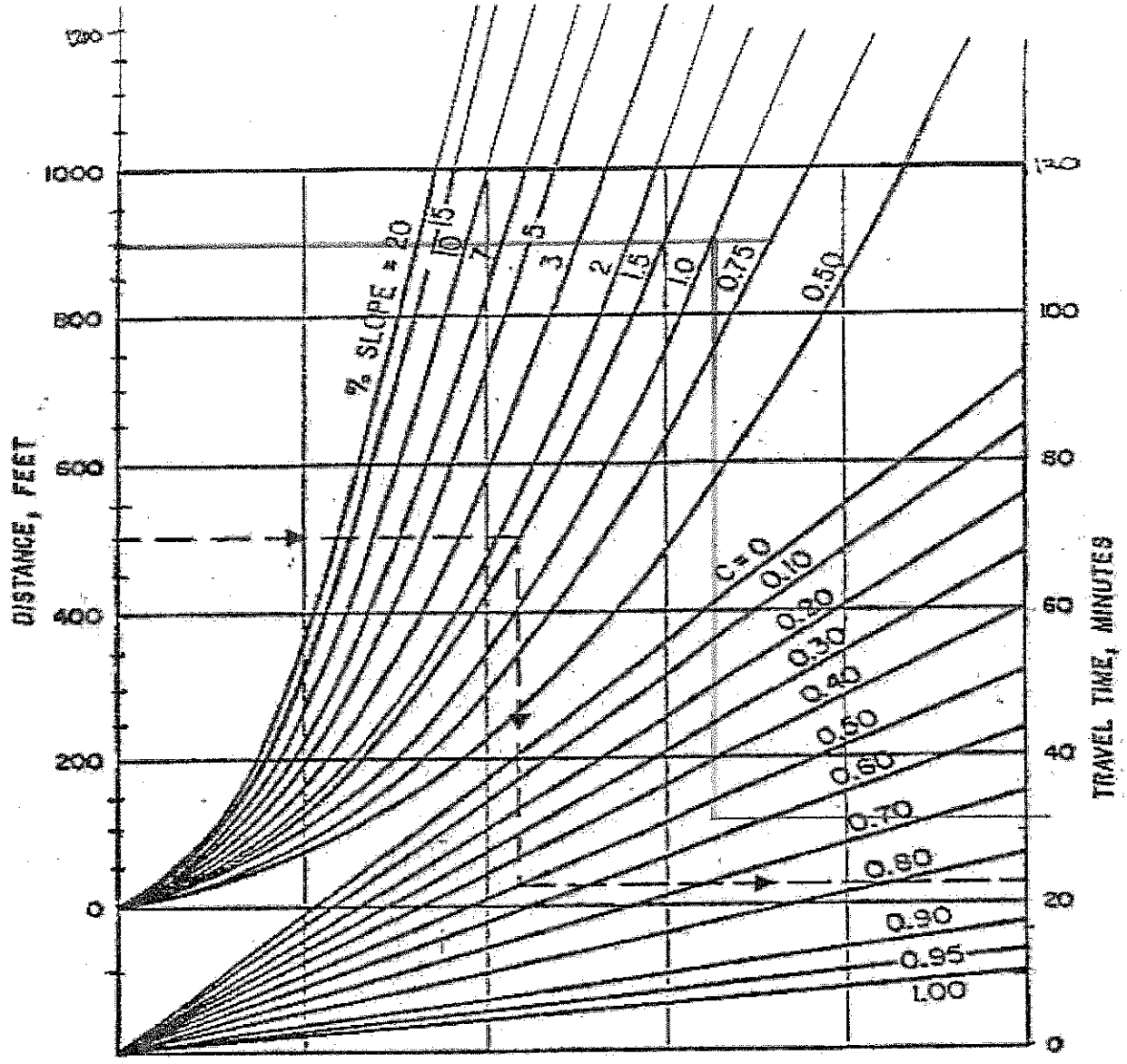
C = 0.5

Slope = ≤ 1%

Travel Time (T_c) = 34

Rainfall Intensity = 1.19

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D4 (25-yr.)

Length = 893 ft.

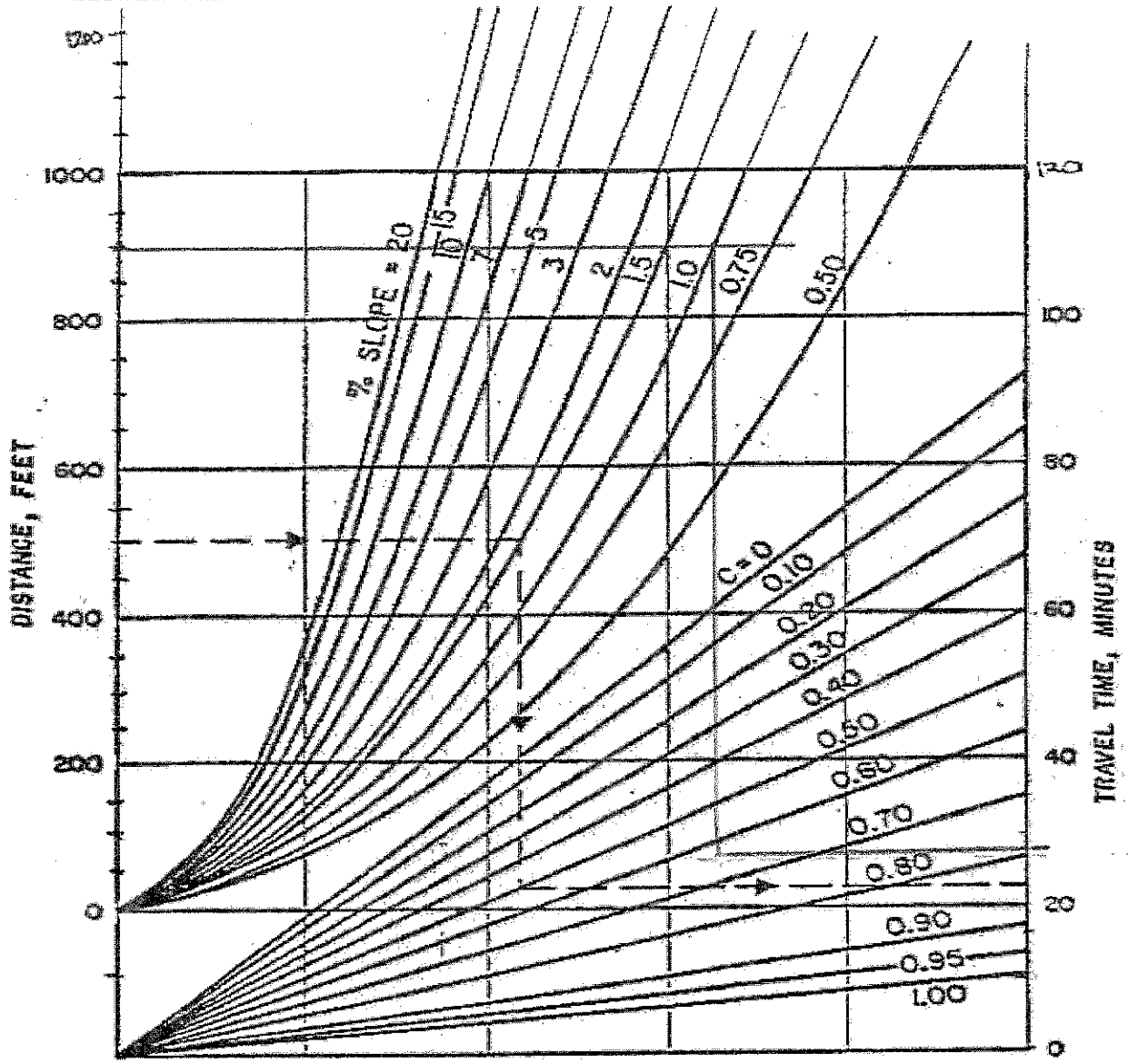
C = 0.54

Slope = ≤ 1%

Travel Time (Tc) = 32

Rainfall Intensity = 1.50

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



D4 (100 yr.)

Length = 893 ft.

C = 0.63

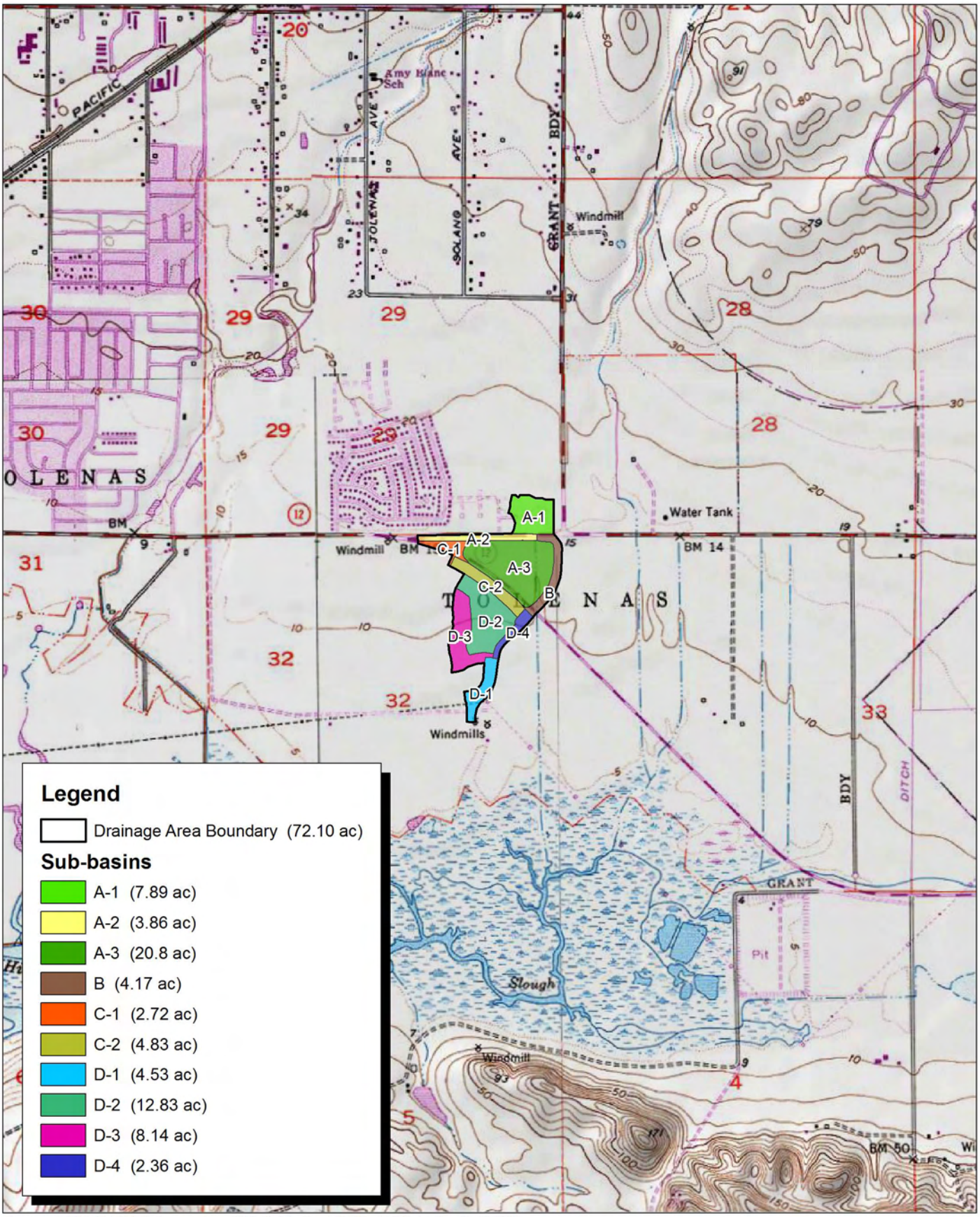
Slope = ≤ 1%

Travel Time (Tc) = 28

Rainfall Intensity = 2.09

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

	Existing Condtions				W/ Project			
	15 year	*25 year*	*100 year*		*15 year*	*25 year*	*100 year*	
Basin A1	4.19	5.62	9.13	*cfs*	4.19	5.62	9.13	*cfs*
Basin A2	7.67	11.12	14.59	*cfs*	7.67	11.12	14.59	*cfs*
Basin A3	7.57	10.15	16.33	*cfs*	27.76	43.51	77.79	*cfs*
Basin B	7.94	11.97	15.71	*cfs*	7.94	11.97	15.71	*cfs*
Basin C1	1.45	1.98	3.17	*cfs*	1.45	1.98	3.17	*cfs*
Basin C2	11.40	9.60	18.96	*cfs*	11.40	9.60	18.96	*cfs*
Basin D1	2.23	2.99	4.95	*cfs*	2.23	2.99	4.95	*cfs*
Basin D2	6.21	8.35	14.08	*cfs*	6.21	8.35	14.08	*cfs*
Basin D3	4.06	5.35	9.02	*cfs*	4.06	5.35	9.02	*cfs*
Basin D4	1.24	1.69	2.78	*cfs*	1.24	1.69	2.78	*cfs*
Total/Outlet	53.95	68.82	108.71	*cfs*	74.13	102.18	170.18	*cfs*
Net Difference	**20.19**	**33.36**	**61.46**					
Convergance SR12/Walters Road	40.21	50.43	77.89		60.40	83.79	139.35	
net	**20.19**	**33.36**	**61.46**					



Source: TOPO! USGS Denverton (1980), Elmira (1980), Fairfield South (1980), Fairfield North (1980); and MBA GIS (2007).

NORTH

2,000 1,000 0 2,000 Feet

Michael Brandman Associates

Exhibit 11
Drainage Sub-Basin Map
Topographic Base

Sub Area A1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	4.187	Qp	5.621245	Qp	9.134375
C	0.5	C	0.535	C	0.625
I	1.06	I	1.33	I	1.85
A	7.9	A	7.9	A	7.9

Rational Method

Qp=CIA

Qp	peak flow rate (cfs)
C	runoff coefficient (dimensional units)
I	rainfall intensity (in/hr)
A	drainage area (aces)

Inputs

Table 1	single family, avg. slope of < 1 % = 0.5
	Figure 1/Table 2 - Travel Time For Overland Flow

Sub Area A2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	7.66935	Qp	11.115	Qp	14.586
C	0.95	C	1	C	1
I	2.07	I	2.85	I	3.74
A	3.9	A	3.9	A	3.9

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	paved area = 0.95
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area A3

Existing Conditions 15 year		Existing Conditions 25 year	(15yr C * 1.2	Existing Conditions 100 yea	(15yr C * 1.2	w/ Project 15 year		w/ Project 25 year	(15yr C * 1.25)	w/ Project 100 year	(15yr C * 1.25)
Qp	7.5712	Qp	10.149	Qp	16.328	Qp	27.7576	Qp	43.51048	Qp	77.792
C	0.4	C	0.428	C	0.5	C	0.85	C	0.9095	C	1
I	0.91	I	1.14	I	1.57	I	1.57	I	2.3	I	3.74
A	20.8	A	20.8	A	20.8	A	20.8	A	20.8	A	20.8

Rational Method

Qp=CIA

Inputs

Qp	peak flow rate (cfs)	
C	runoff coefficient (dimensional Table 1	Existing Conditions - Undeveloped Open Space, including pasture > 2 % slope = 0.4
I	rainfall intensity (in/hr)	W/ Project - Commerical = 0.85
A	drainage area (aces)	Figure 1/Table 2 - Travel Time For Overland Flow

Sub Area C1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	1.4472	Qp	1.976076	Qp	3.1725
C	0.4	C	0.428	C	0.5
I	1.34	I	1.71	I	2.35
A	2.7	A	2.7	A	2.7

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	Undeveloped Open Space, including pasture > 2 % slope = 0.4
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area C2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	11.4	Qp	9.6	Qp	18.96

Note: Subbasin C2 was subdivided into two smaller basins (C2a and C2b) due to the a drainage length that is > 1200 ft.

Rational Method

Qp=CIA			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	paved area = 0.95
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)		(C2a)
Qp	5.7	Qp	4.8	Qp	9.48	
C	0.95	C	1	C	1	
I	2.5	I	2	I	3.95	
A	2.4	A	2.4	A	2.4	

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)		(C2b)
Qp	5.7	Qp	4.8	Qp	9.48	
C	0.95	C	1	C	1	
I	2.5	I	2	I	3.95	
A	2.4	A	2.4	A	2.4	

Sub Area D1

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	2.2275	Qp	2.9853	Qp	4.95
C	0.5	C	0.535	C	0.625
I	0.99	I	1.24	I	1.76
A	4.5	A	4.5	A	4.5

Rational Method

Qp=CIA			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area D2

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	6.208	Qp	8.35456	Qp	14.08
C	0.5	C	0.535	C	0.625
I	0.97	I	1.22	I	1.76
A	12.8	A	12.8	A	12.8

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

Sub Area D3

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	4.059	Qp	5.35214	Qp	9.02
C	0.5	C	0.535	C	0.625
I	0.99	I	1.22	I	1.76
A	8.2	A	8.2	A	8.2

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		

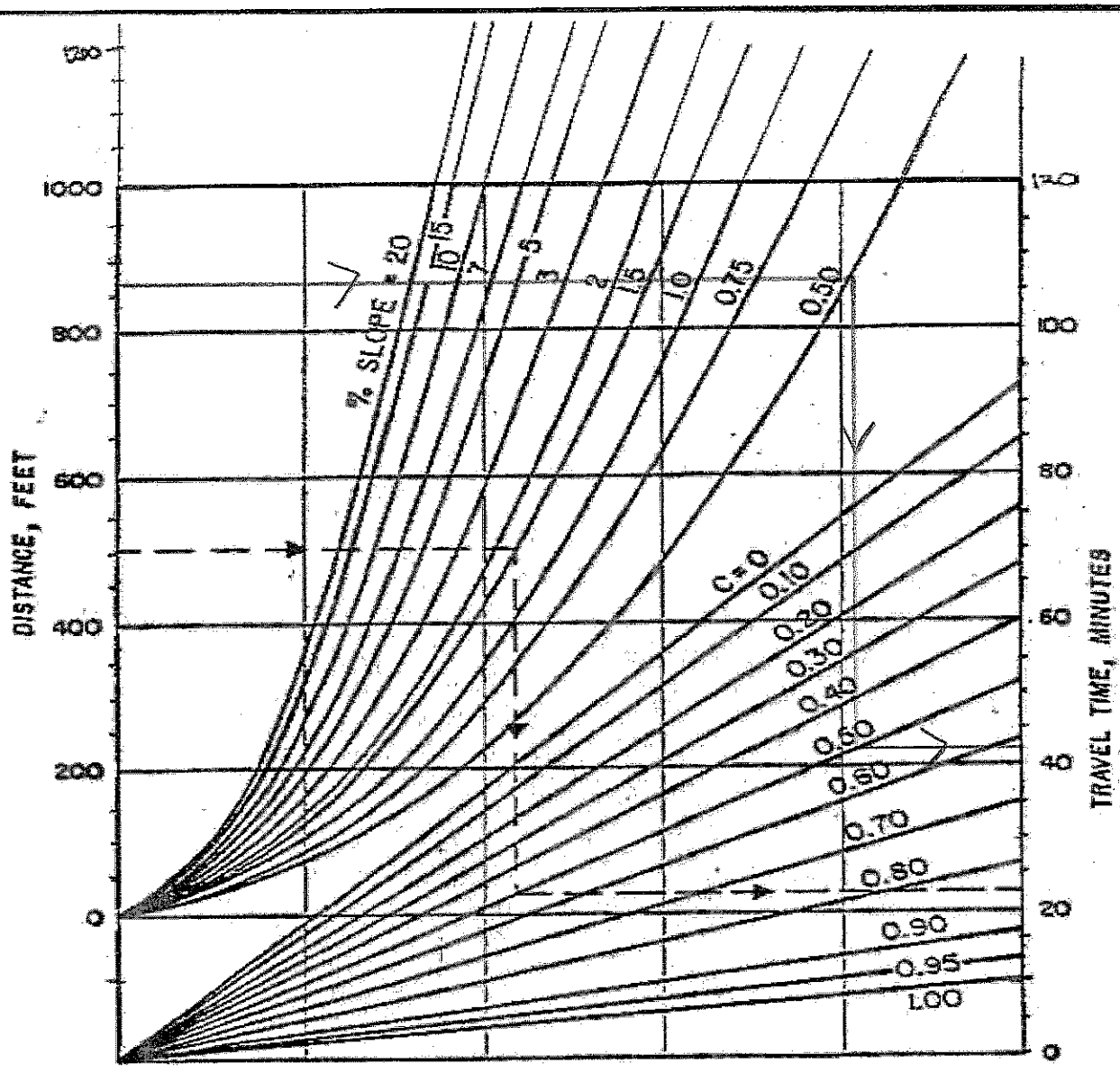
Sub Area D4

Existing Conditions 15 year		Existing Conditions 25 year (15yr C * 1.07)		Existing Conditions 100 year (15yr C * 1.25)	
Qp	1.236	Qp	1.69488	Qp	2.775
C	0.5	C	0.535	C	0.625
I	1.03	I	1.32	I	1.85
A	2.4	A	2.4	A	2.4

Rational Method

Qp=CIA

			Inputs
Qp	peak flow rate (cfs)		
C	runoff coefficient (dimensional units)	Table 1	single family, avg. slope of < 2 % = 0.5
I	rainfall intensity (in/hr)		Figure 1/Table 2 - Travel Time For Overland Flow
A	drainage area (aces)		



Basin A1 (15 yr.)

Length - 865 ft.

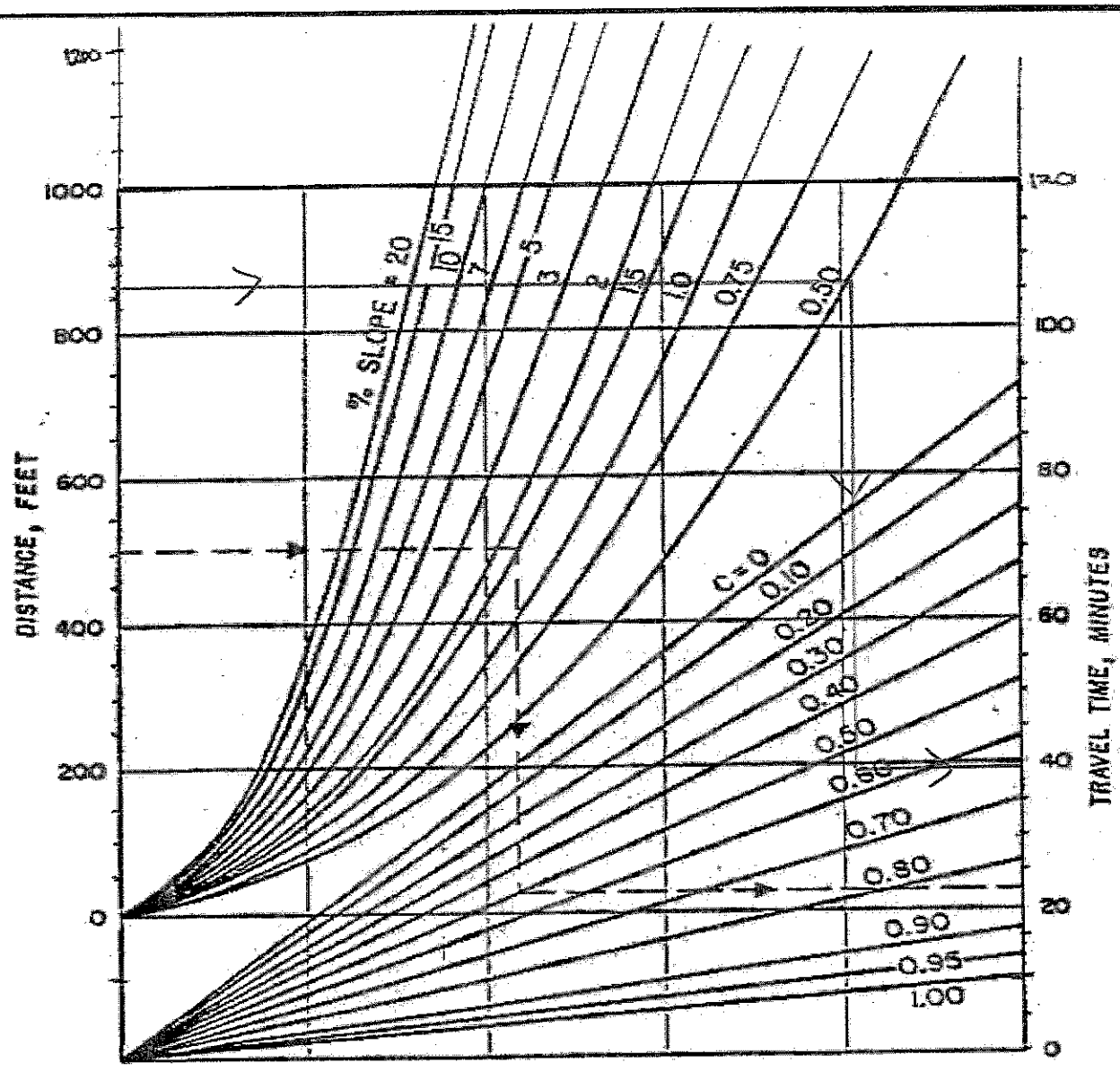
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 42

Rainfall Intensity = 1.06
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A1 (25 ac)

Length - 865 ft.

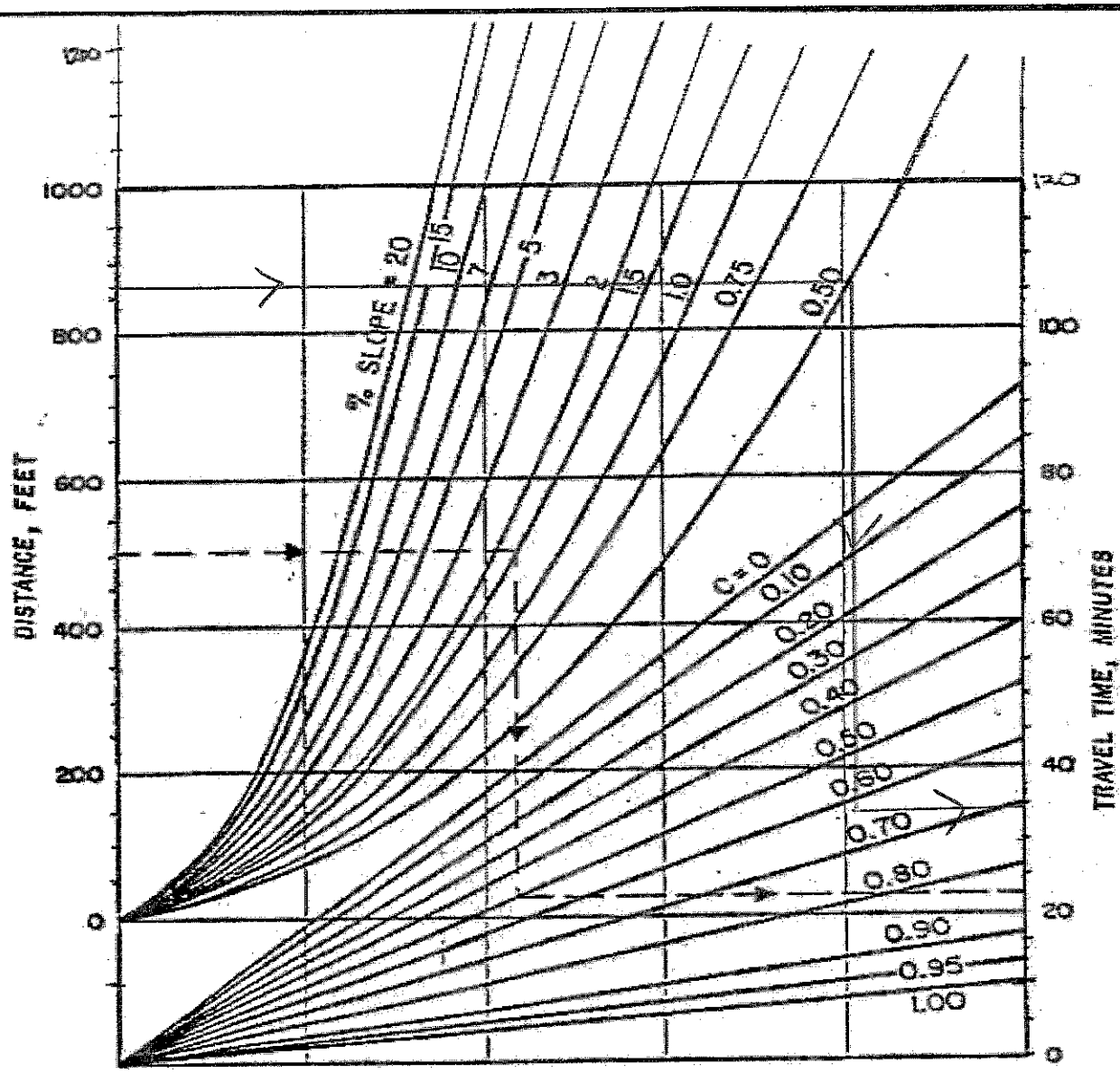
Curve - 0.535

Slope - 0.5%

Travel Time (T_c) = 39

Rainfall Intensity = 1.33
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A1 (100 yr)

Length - 865 ft.

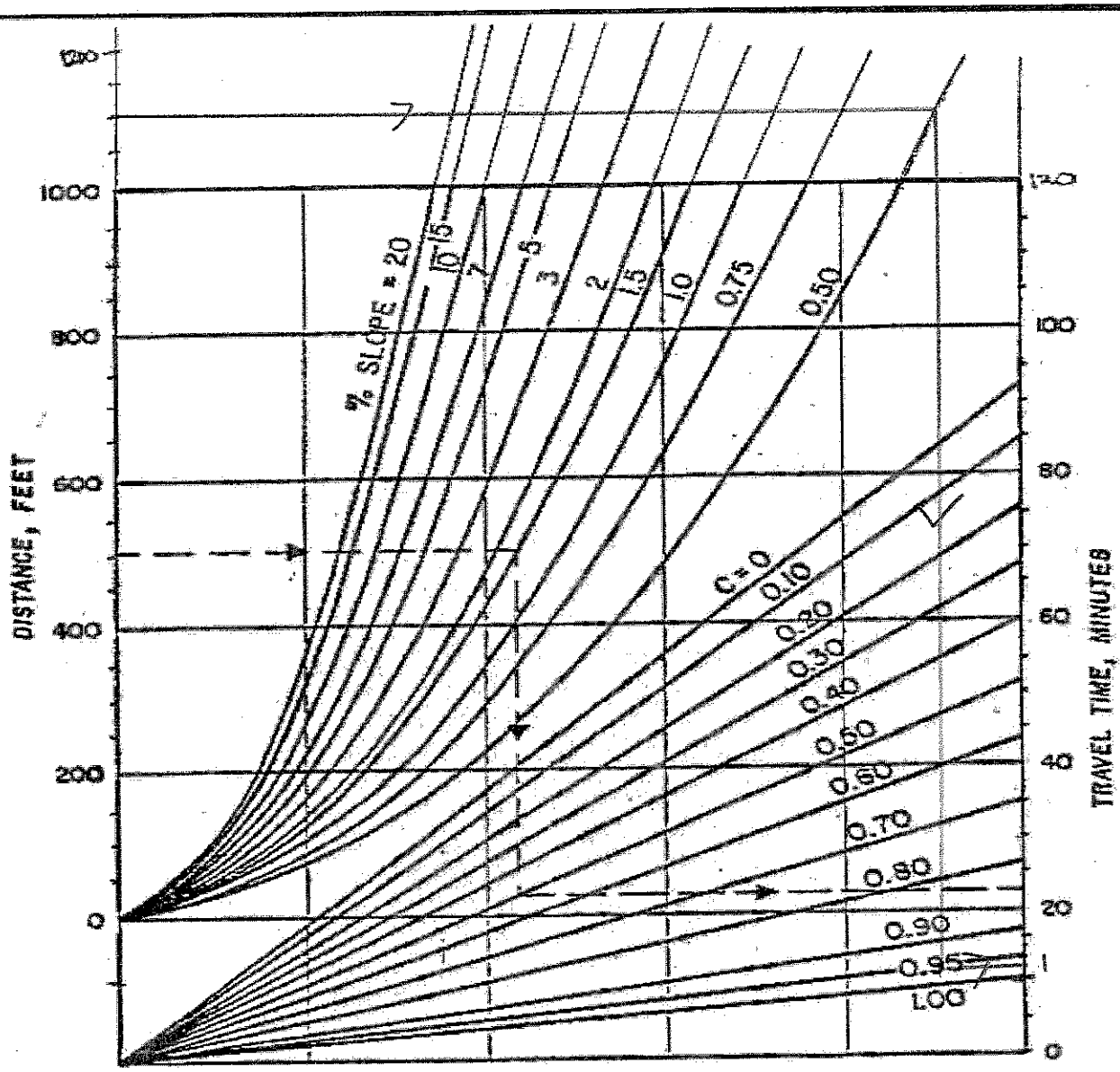
Curve - 6.25

Slope - 0.5%

Travel Time (T_c) = 35

Rainfall Intensity = 1.85
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A2 (15 yr.)

Length - 1091 ft.

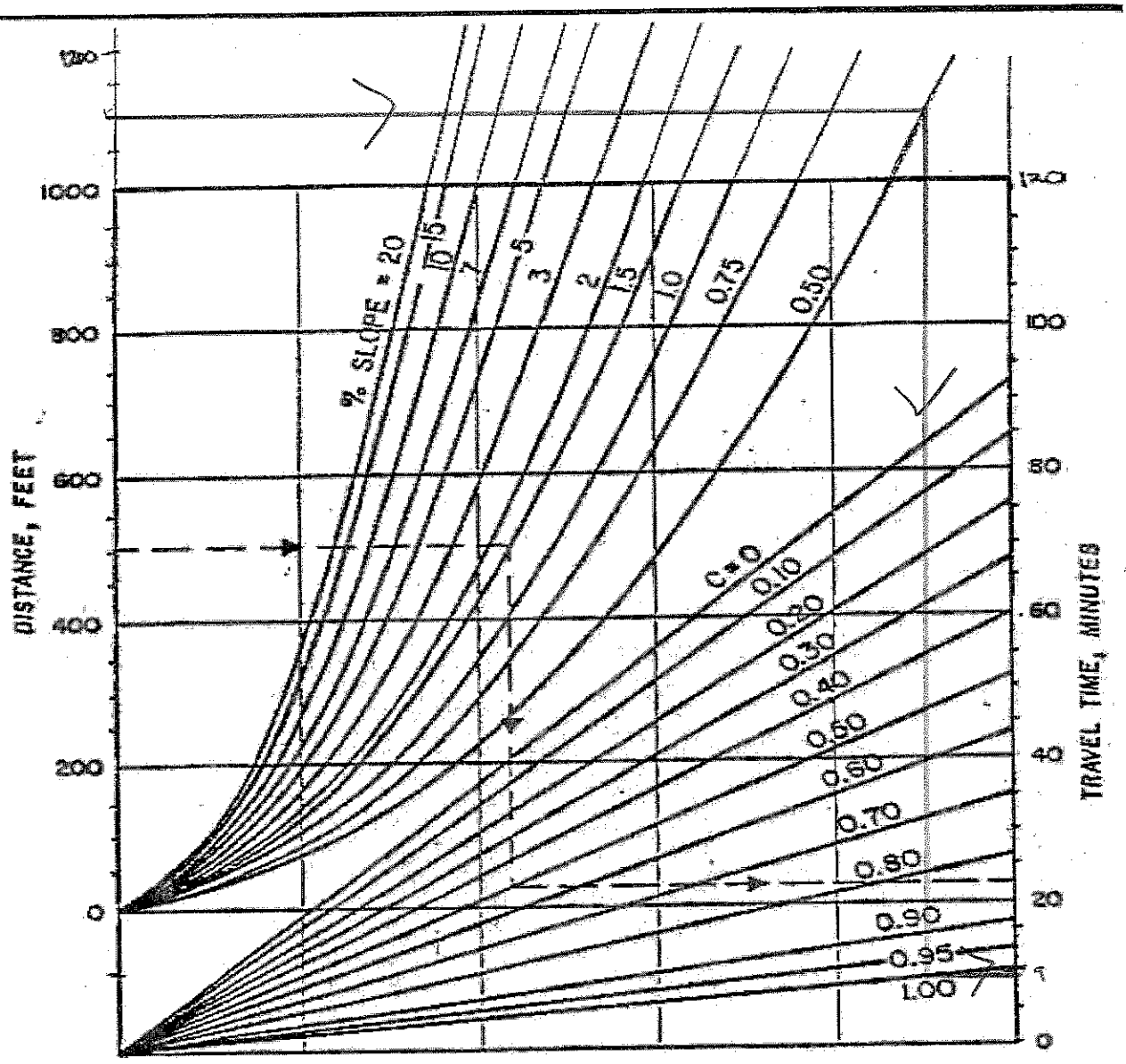
C value - 0.95

Slope - 0.5%

Travel Time (T_c) = 12

Rainfall Intensity = 2.07
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A2 (25-yr)

Length - 1091 ft

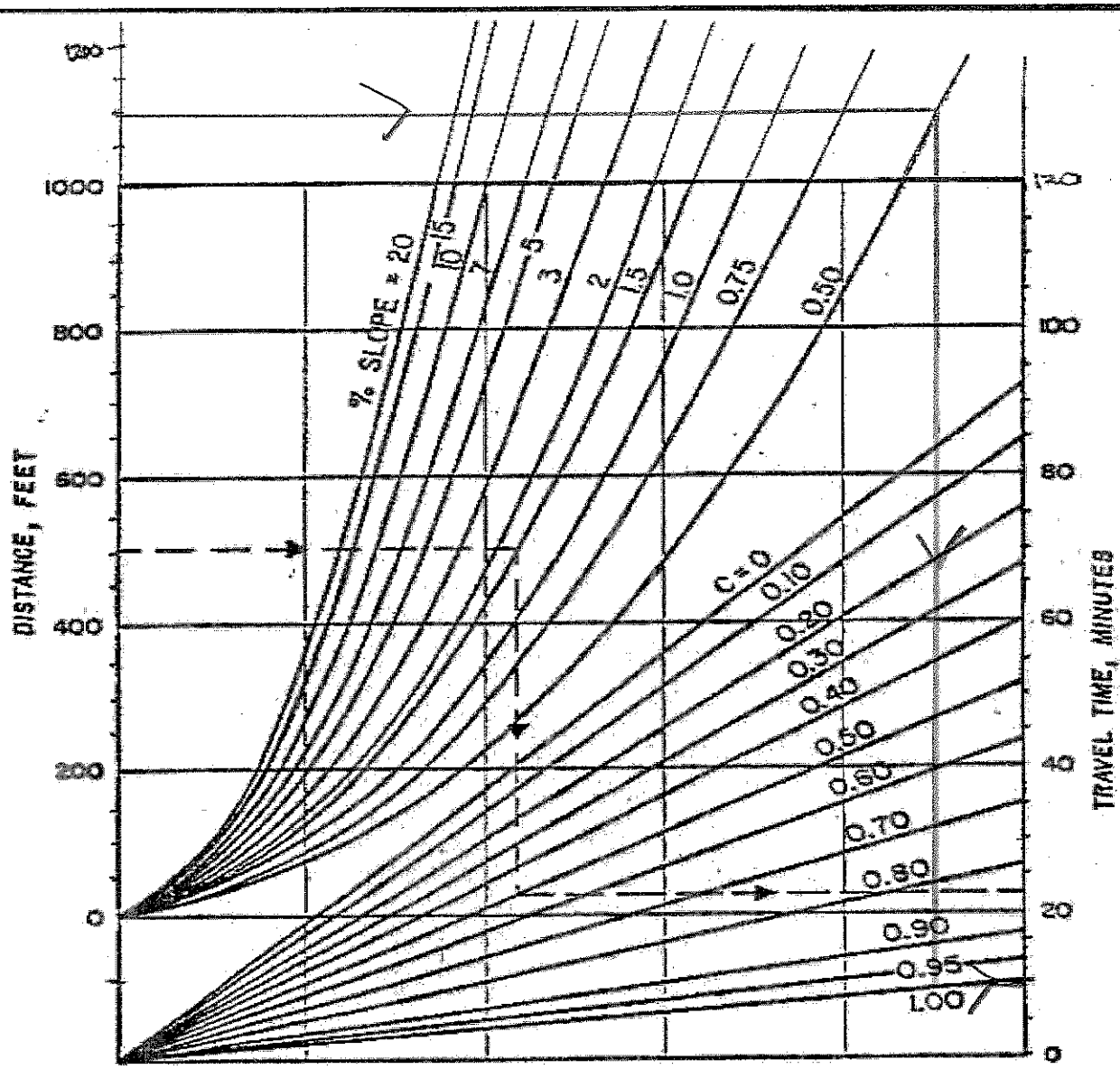
Curve - 1.0

Slope - 0.5%

Travel Time (T_c) = 9

Rainfall Intensity = 2.85
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A2 (100 yr.)

Length - 1,091 ft.

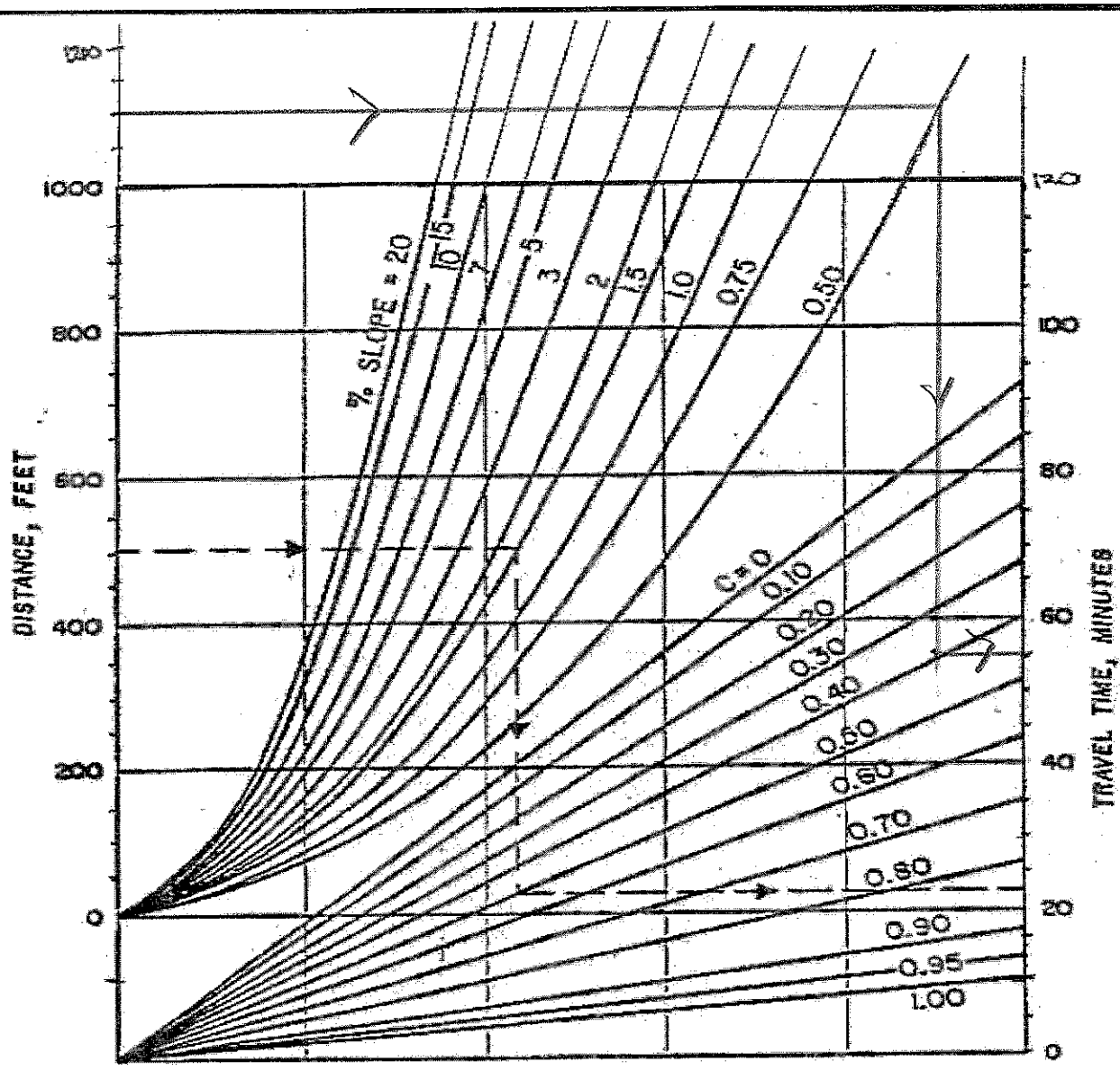
Cvalue - 1.0

Slope - 0.5%

Travel Time (T_c) = 9

Rainfall Intensity = 3.74
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (15-yr.-Existing)

Length - 1097

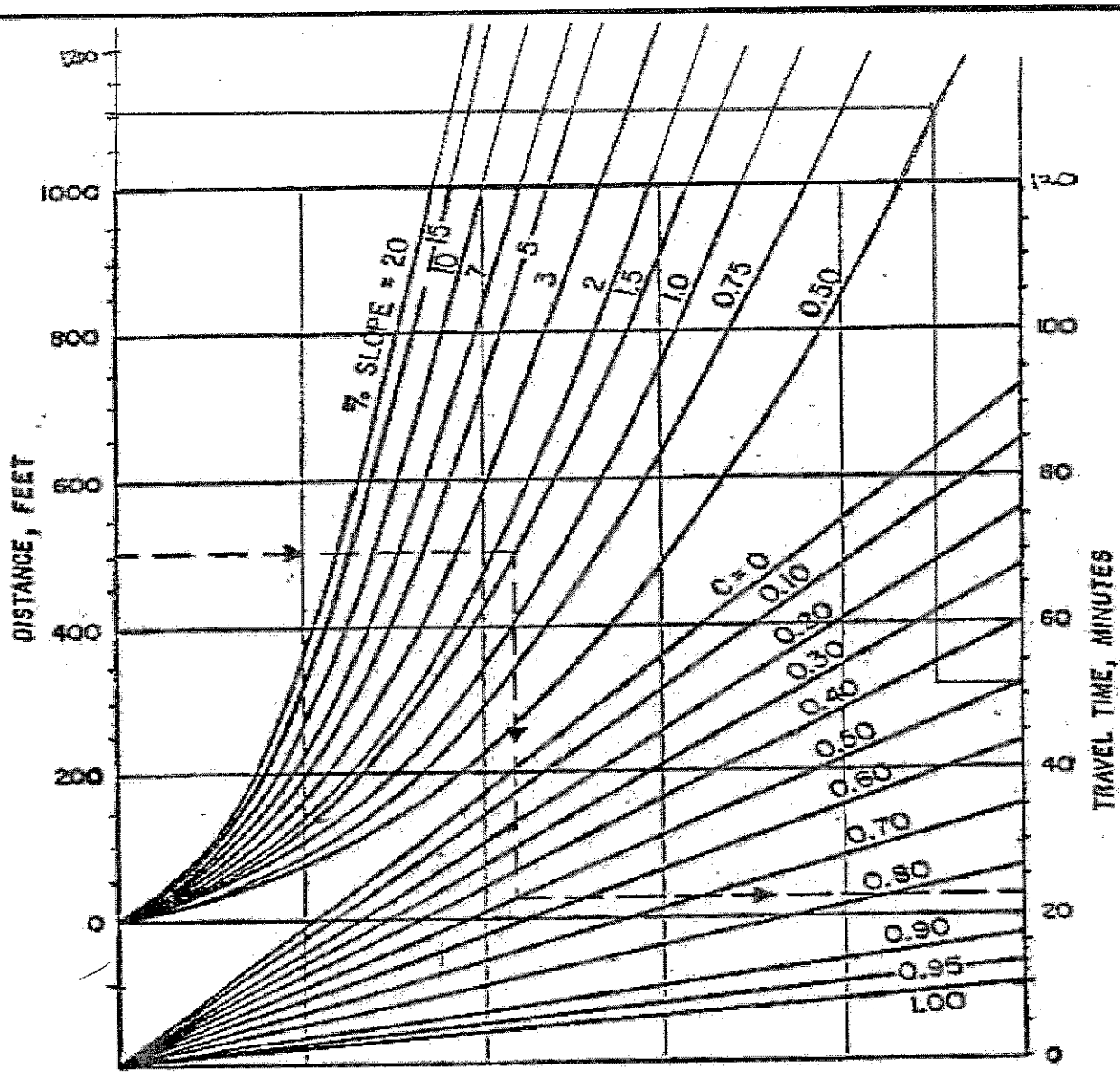
C value - 0.4

Slope - 0.5%

Travel Time (T_c) = 55

Rainfall Intensity = 0.91
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (25-yr- Existing)

Length - 1097

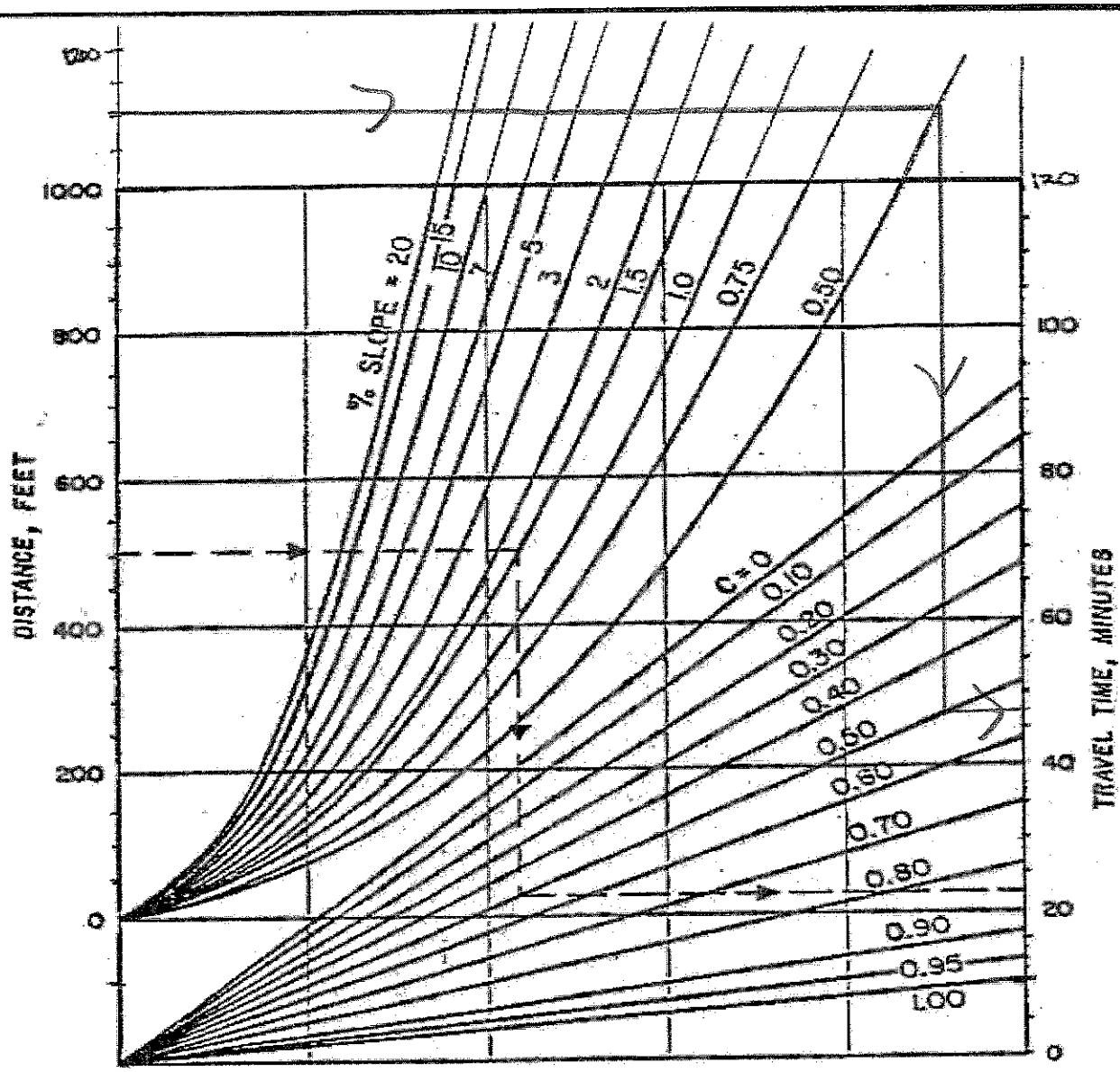
C value - 0.428

Slope - 0.5%

Travel Time (T_c) = 52

Rainfall Intensity = 1.14
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (100 yr - Existing Conditions)

Length - 1097

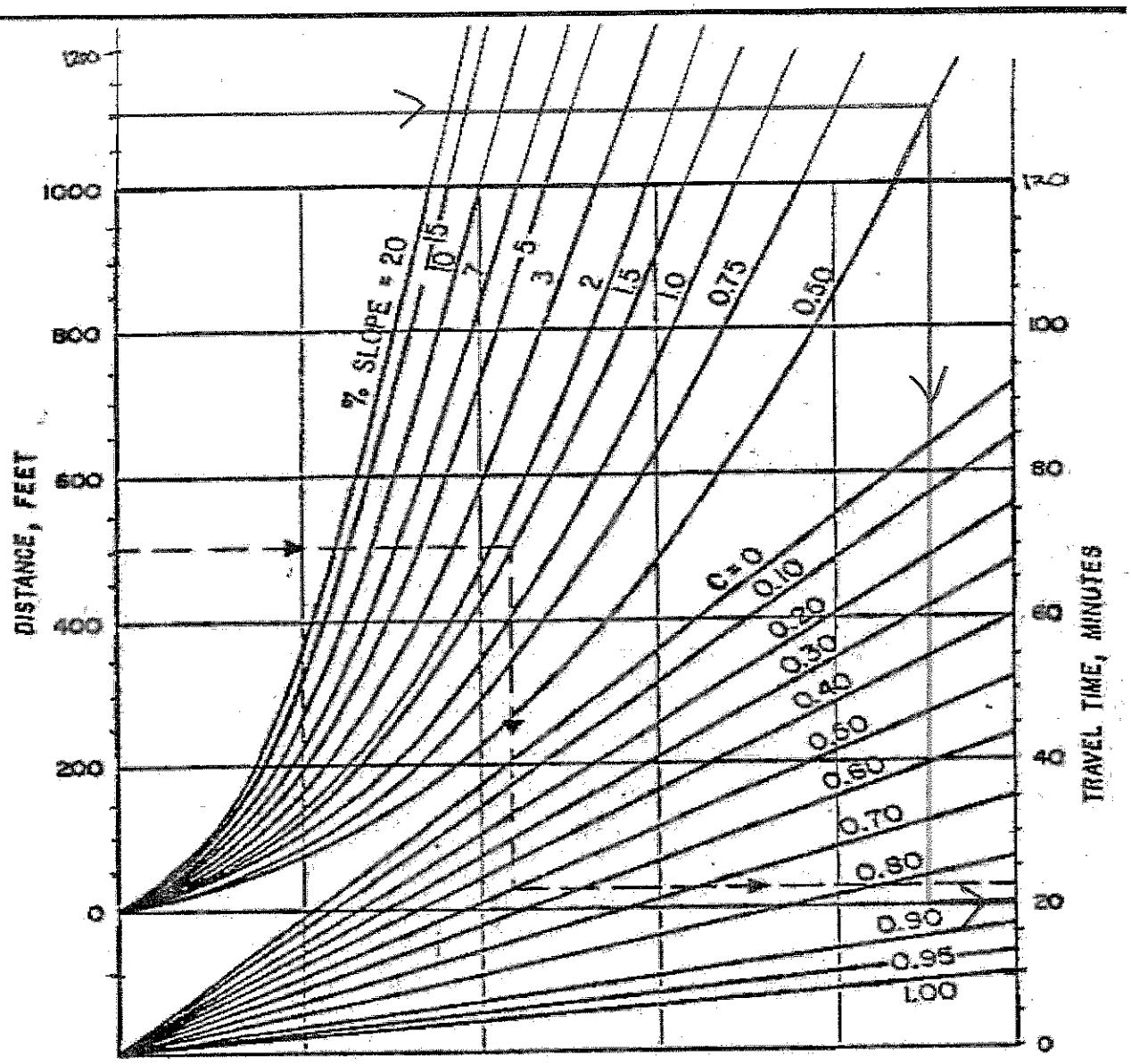
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 47

Rainfall Intensity = 1.57
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (15-yr - Project)

Length - 1097

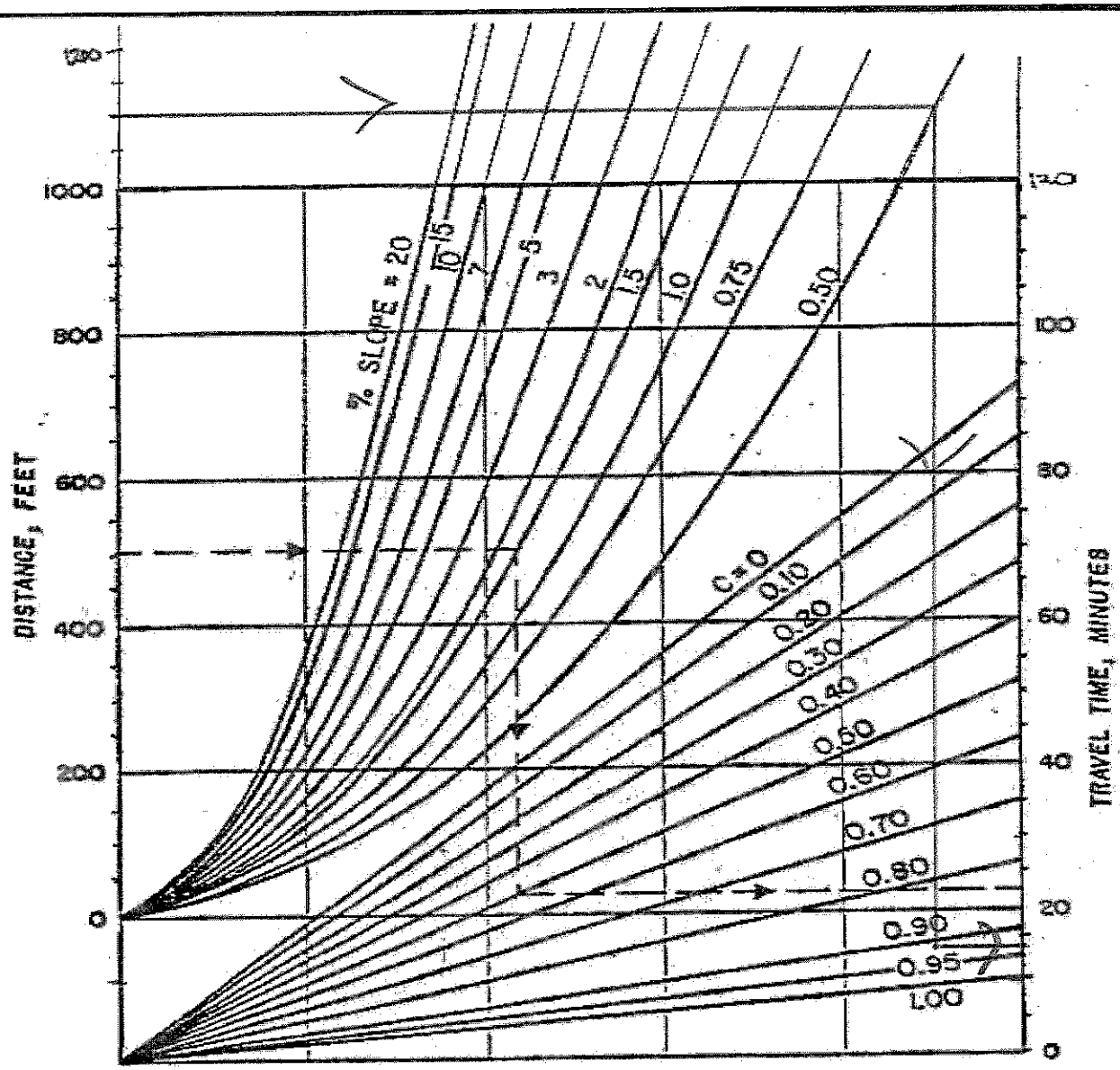
C value - 0.85

Slope - 0.5%

Travel Time (T_c) = 20

Rainfall Intensity = 1.57
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (25 yr - Project)

Length - 1097 ft.

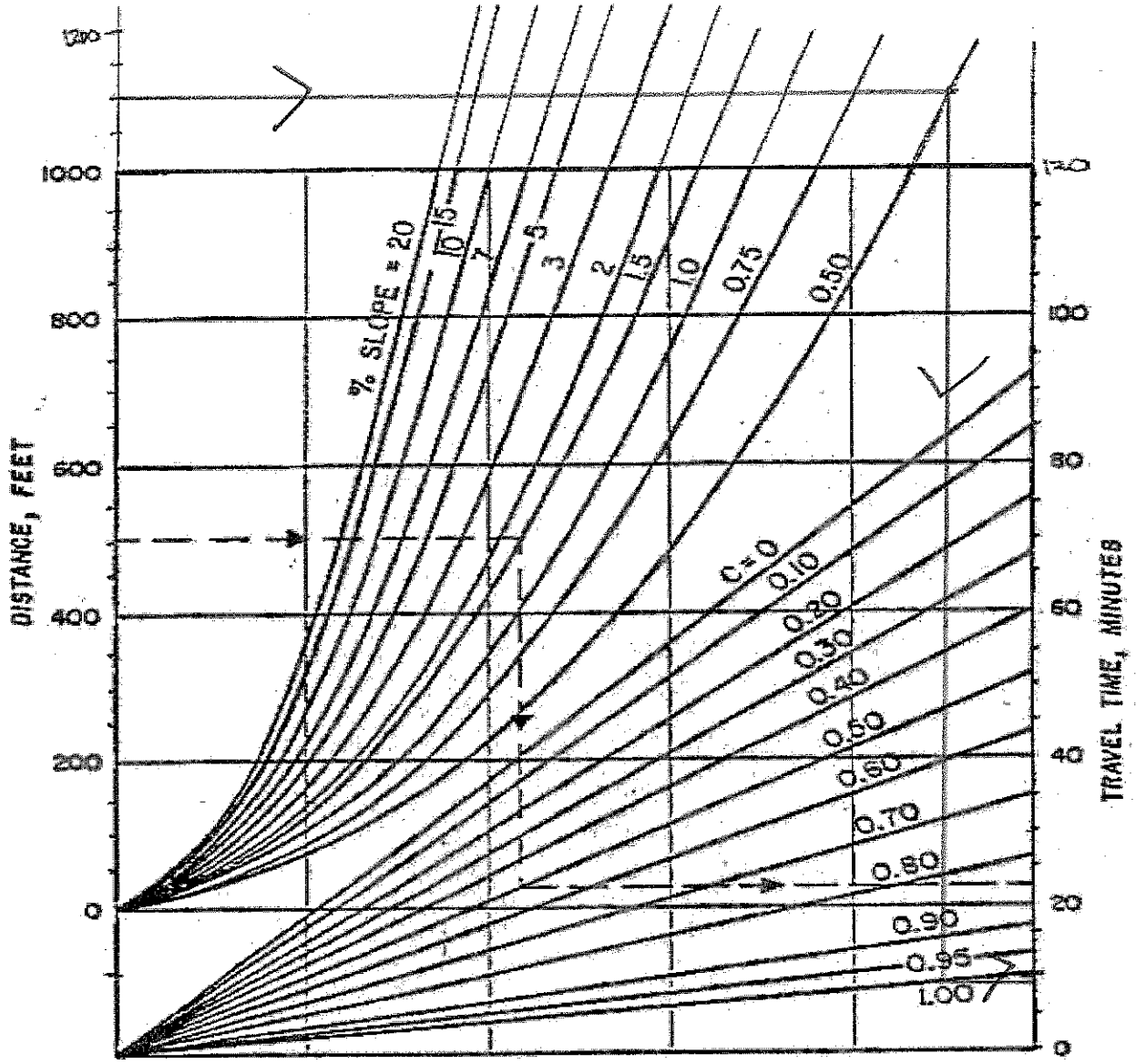
C value - 0.91

Slope - 0.5%

Travel Time (T_c) = 14

Rainfall Intensity = 2.30
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin A3 (100-yr-Project)

Length - 1097 ft.

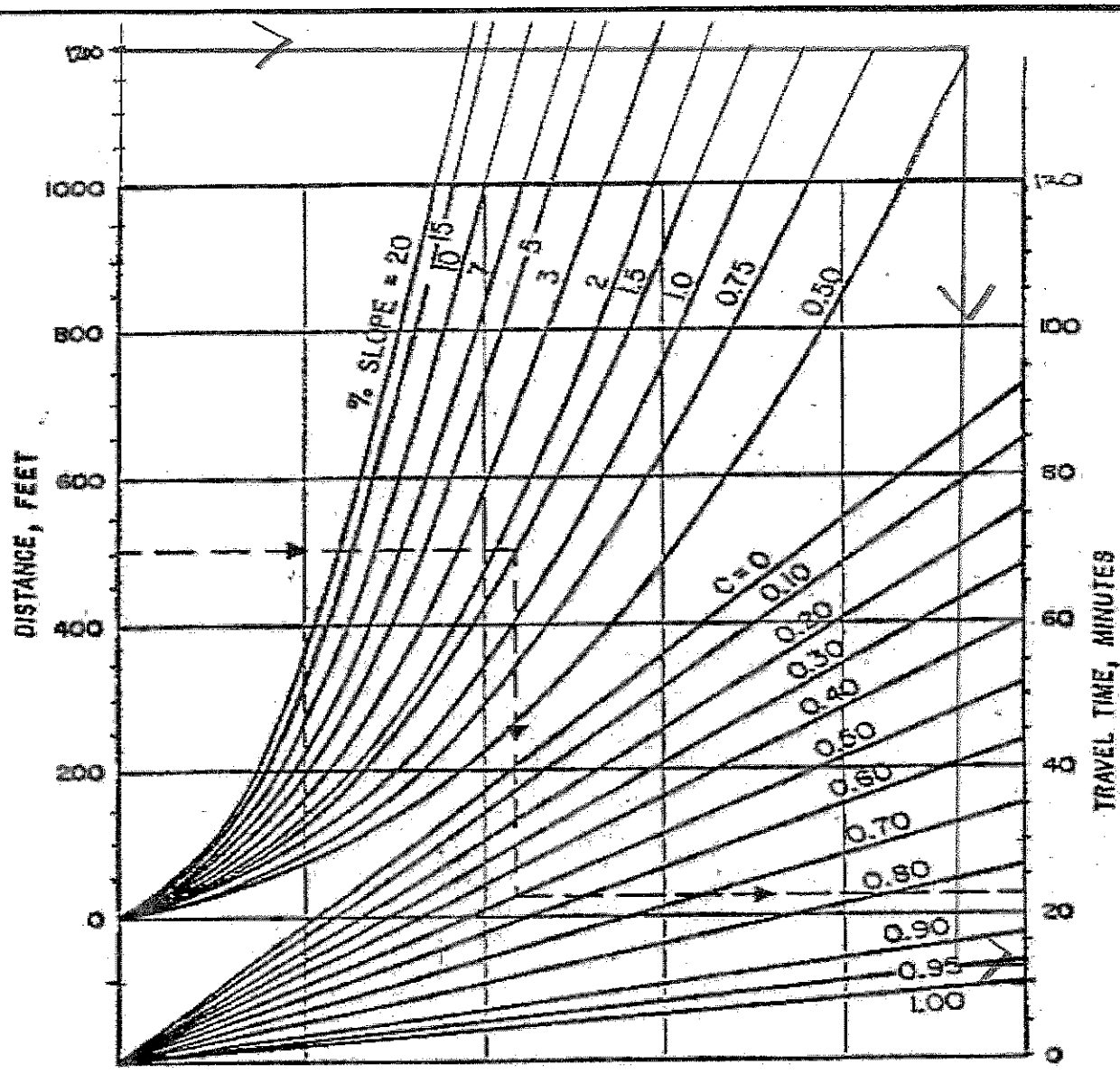
Cvalue - 1.0

Slope - 0.5%

Travel Time (T_c) = 9

Rainfall Intensity = 3.74
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin B (15 yr.)

Length - 1224 ft → 1200

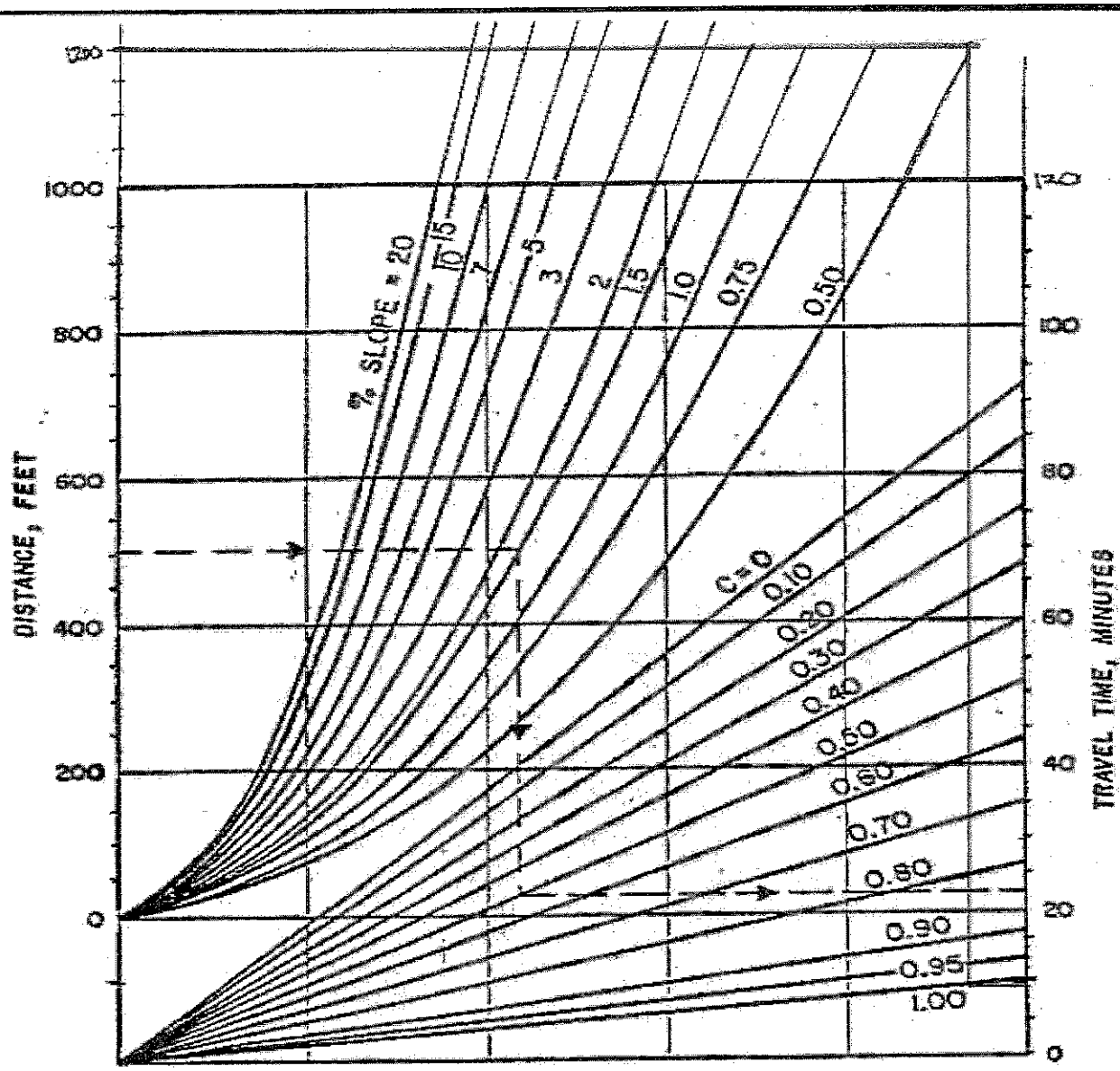
C value - 0.95

Slope - 0.5%

Travel Time (T_c) = 13

Rainfall Intensity = 1.99
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin B (25 yr.)

Length - 1224 ft — 1200

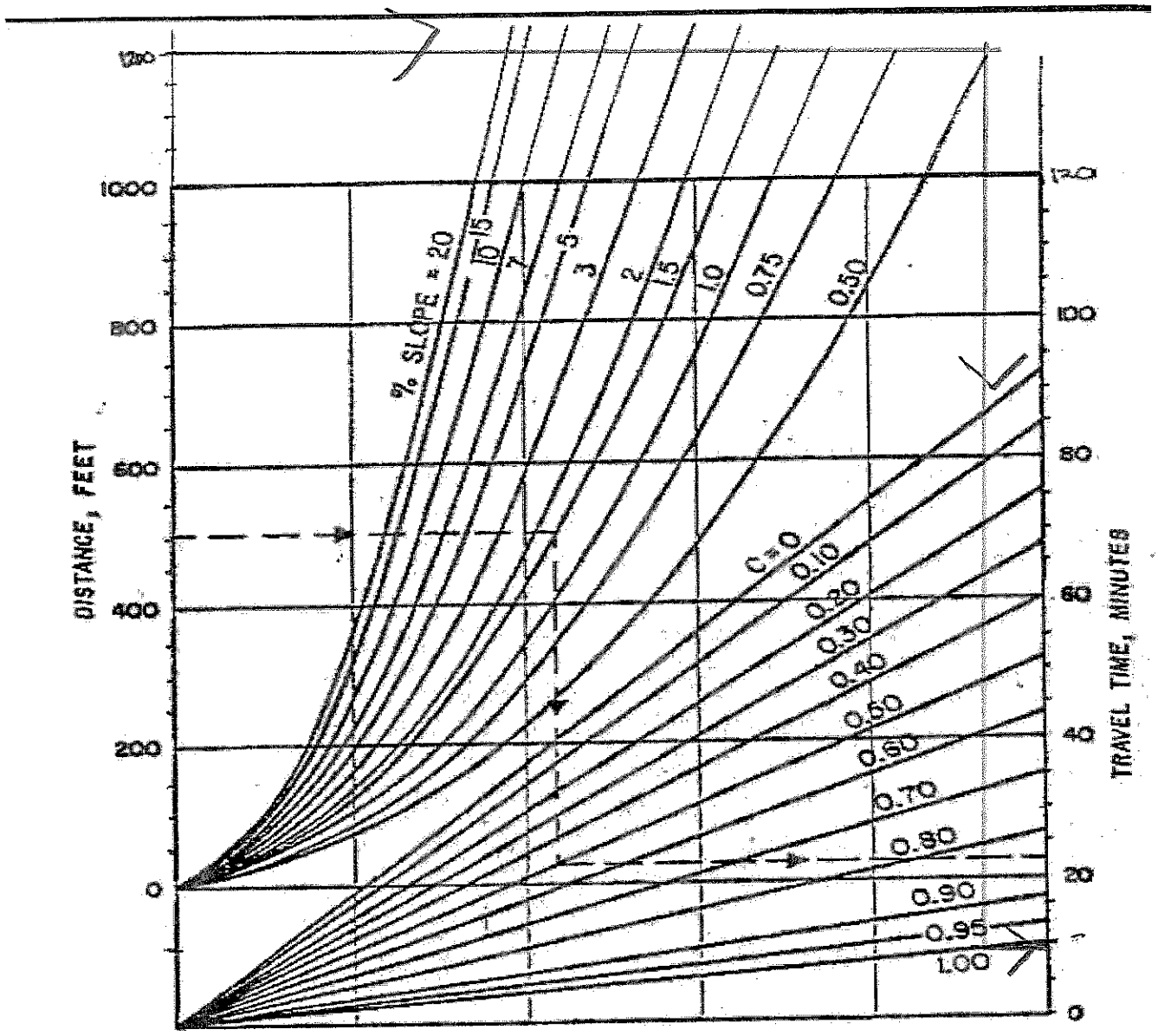
C value - 1.0

Slope - 0.5%

Travel Time (T_c) = 9

Rainfall Intensity = 2.85
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin B (100 yr.)

Length - 1224 ft - 1200

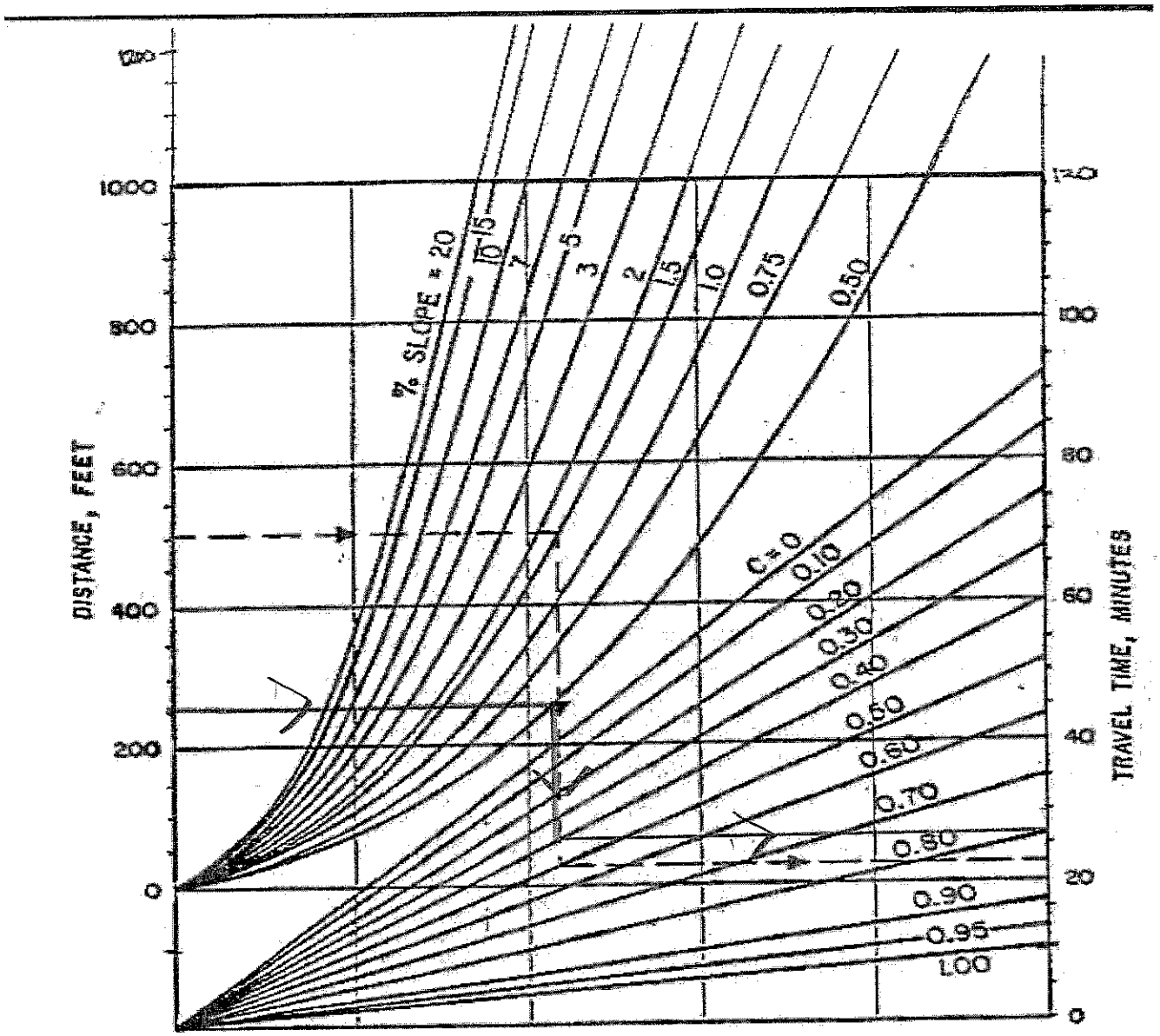
C value - 1.0

Slope - 0.5%

Travel Time (T_c) = 9

Rainfall Intensity = 3.74
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin C1 (15-yr)

Length - 246 ft.

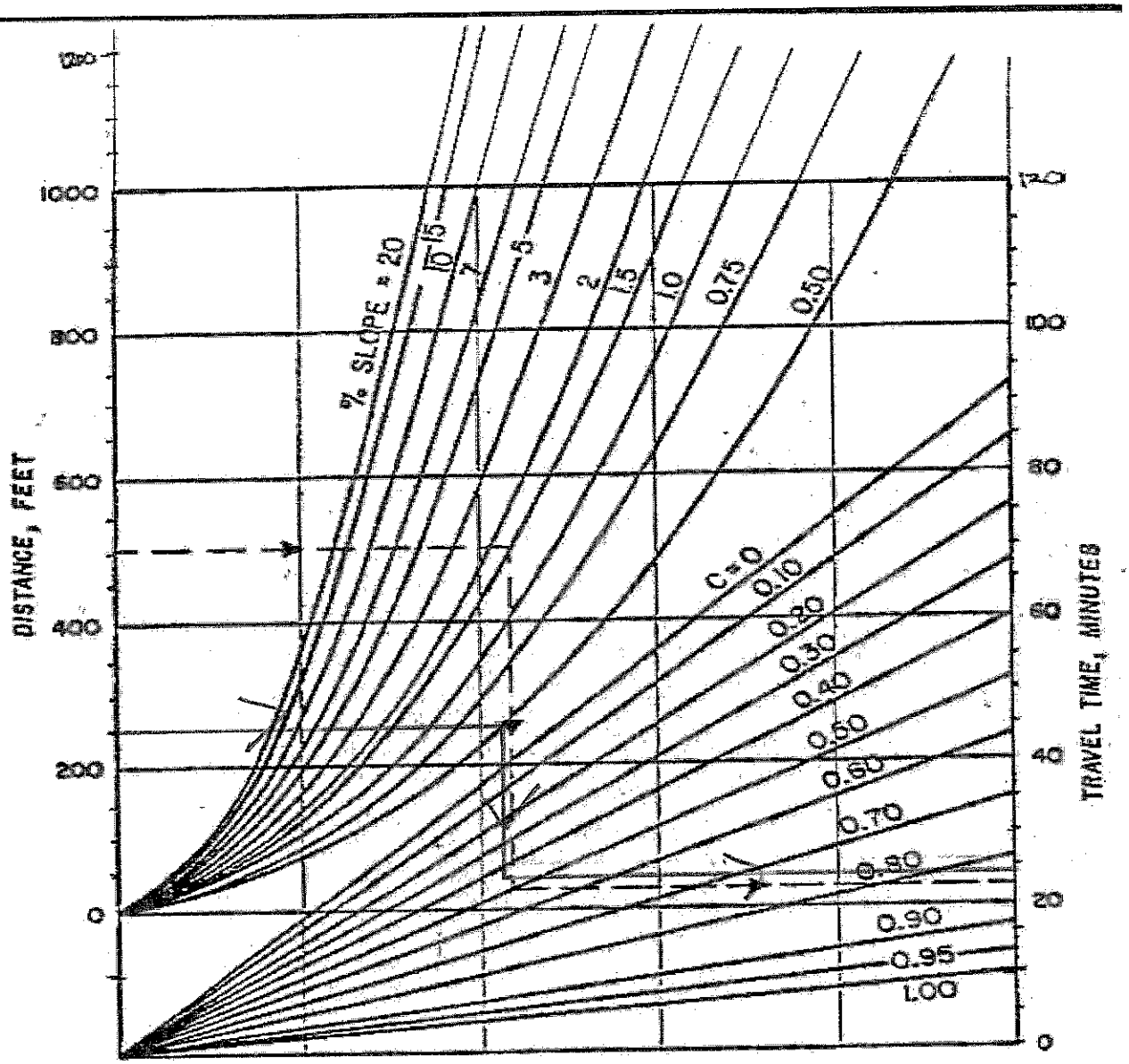
C value - 0.4

Slope - 0.5%

Travel Time (T_c) = 27

Rainfall Intensity = 1.39
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin C1 (25 yr.)

Length - 246 ft

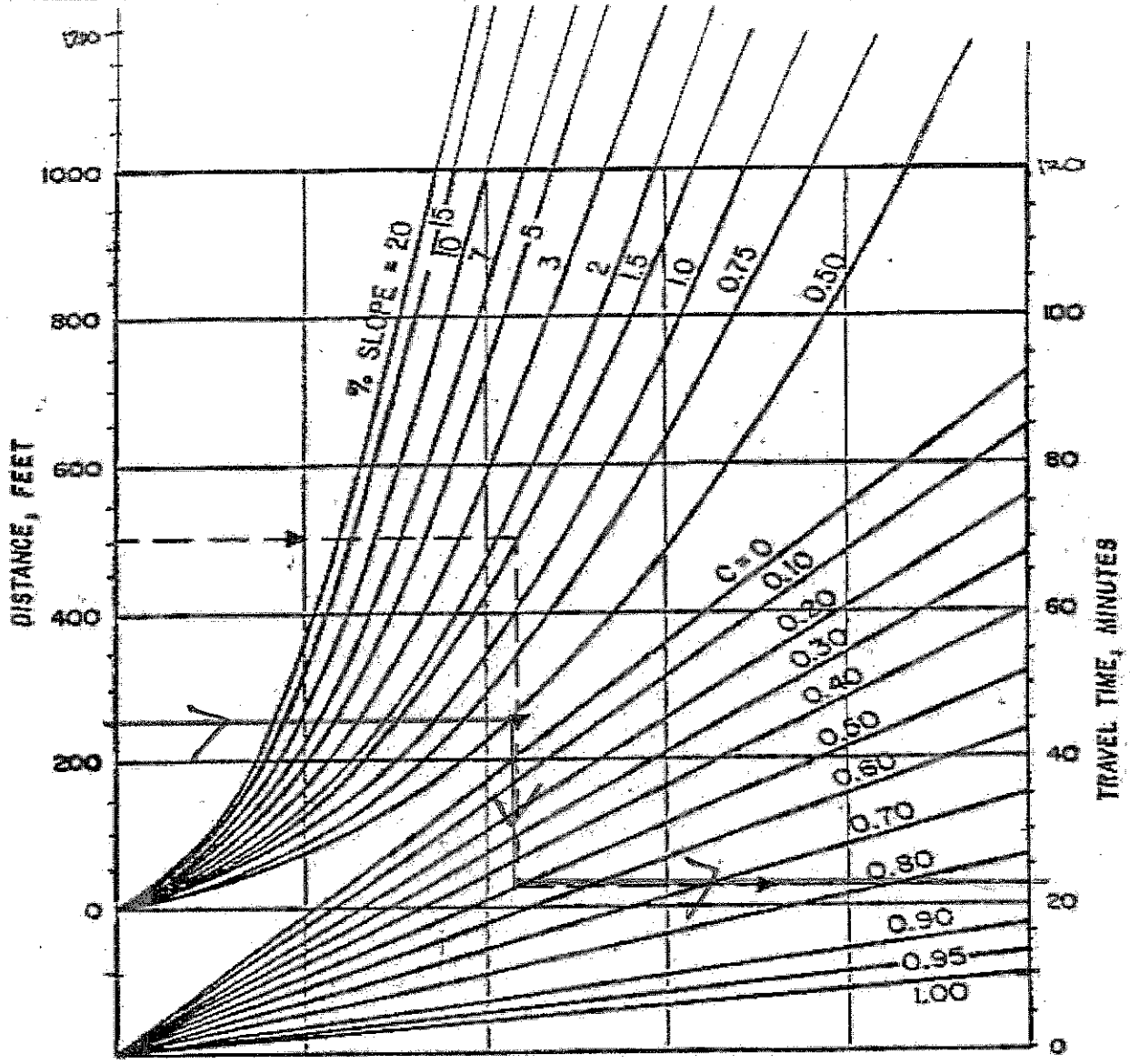
C value - 0.428

Slope - 0.5%

Travel Time (T_c) = 24

Rainfall Intensity = 1.71
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin C1 (100 yr.)

Length - 246 ft

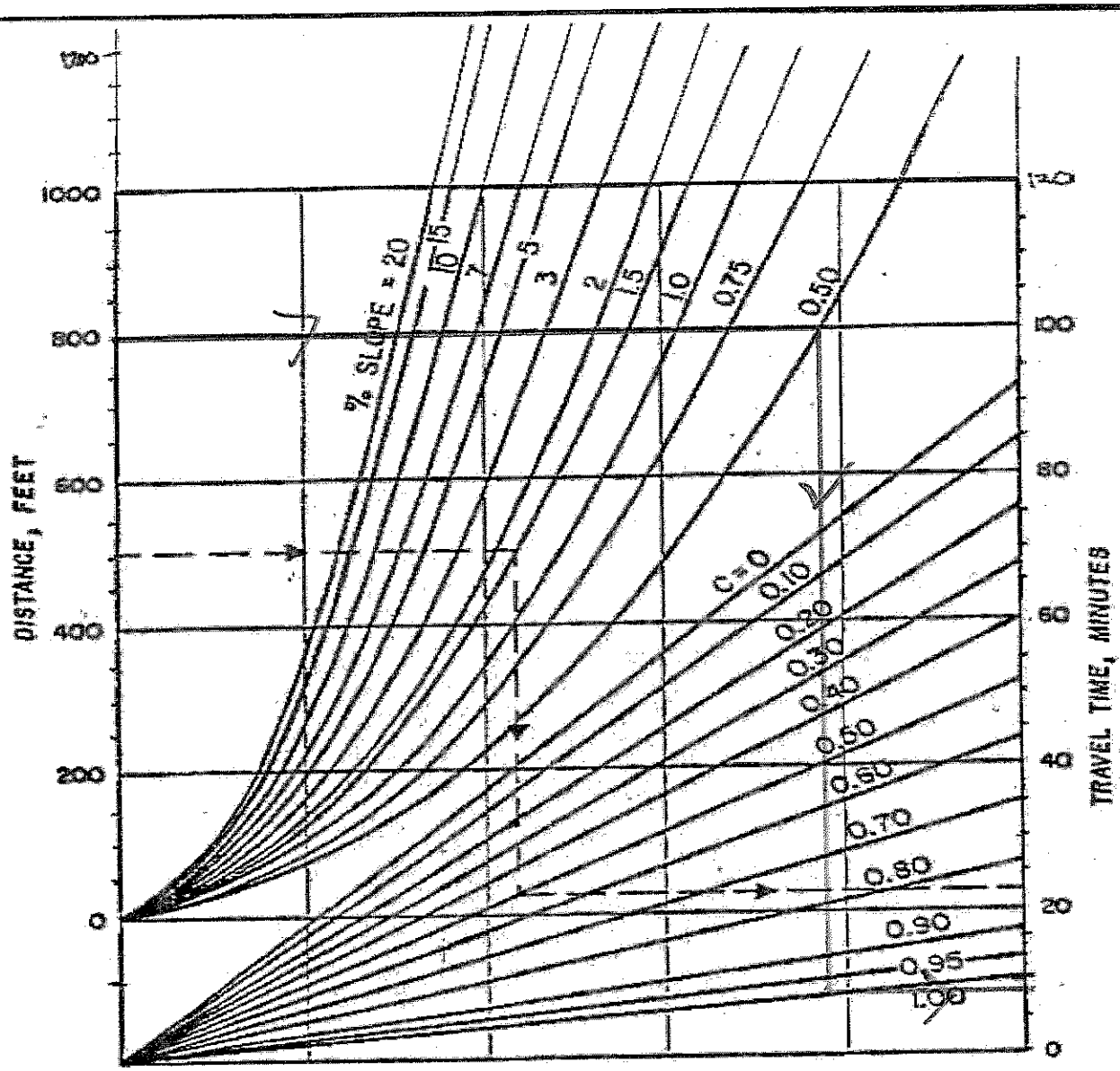
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 22

Rainfall Intensity = 2.35
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin C1a (15yr),

Length - 792

C value - 1.0

Slope - 0.5%

23yr = 2.00

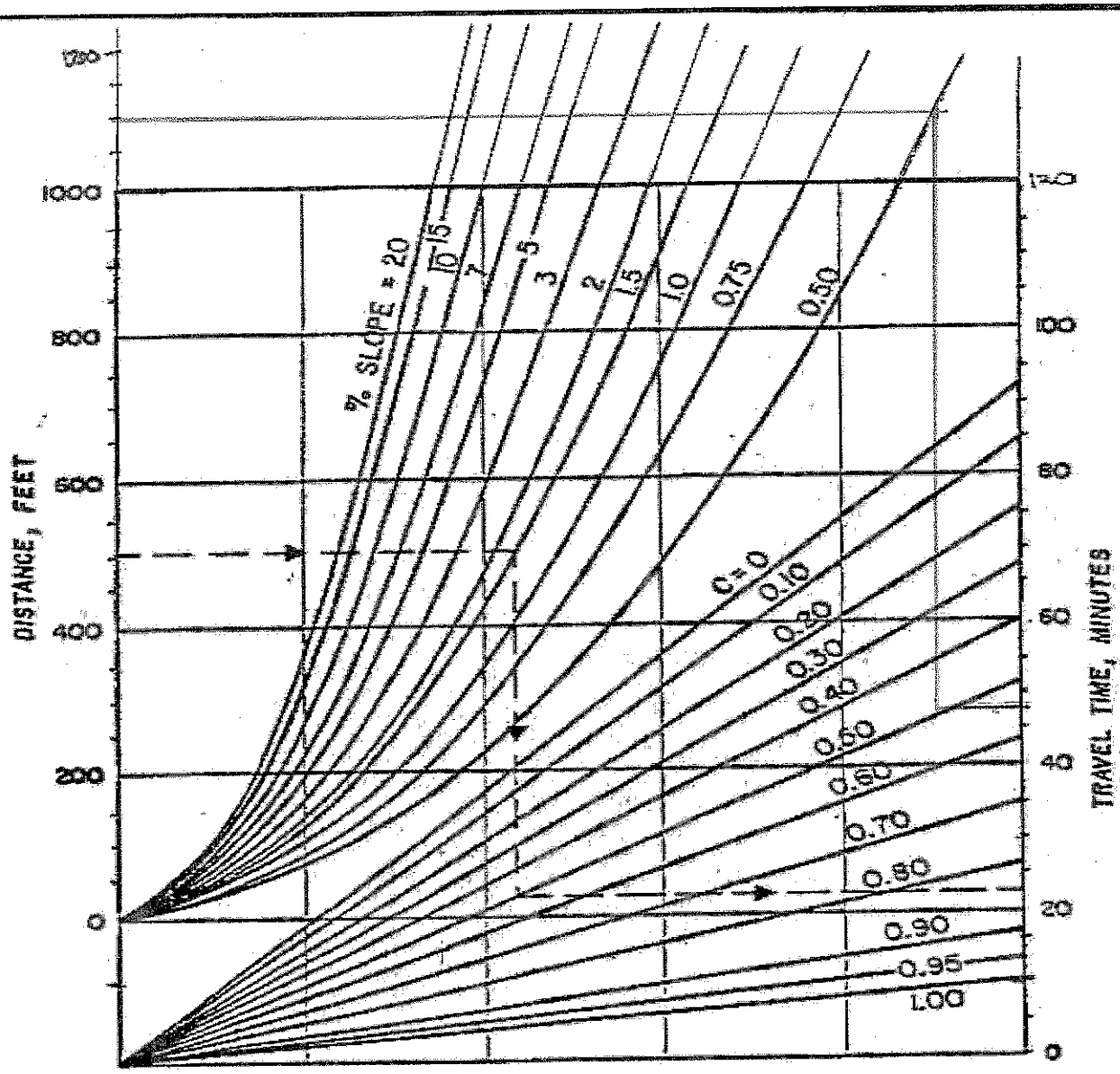
100yr. = 3.95

Travel Time (T_c) = 8

Rainfall Intensity = 2.50
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

Note: Calculations Apply to C1b



Basin D1 (15 yr.)

Length - 1089 ft

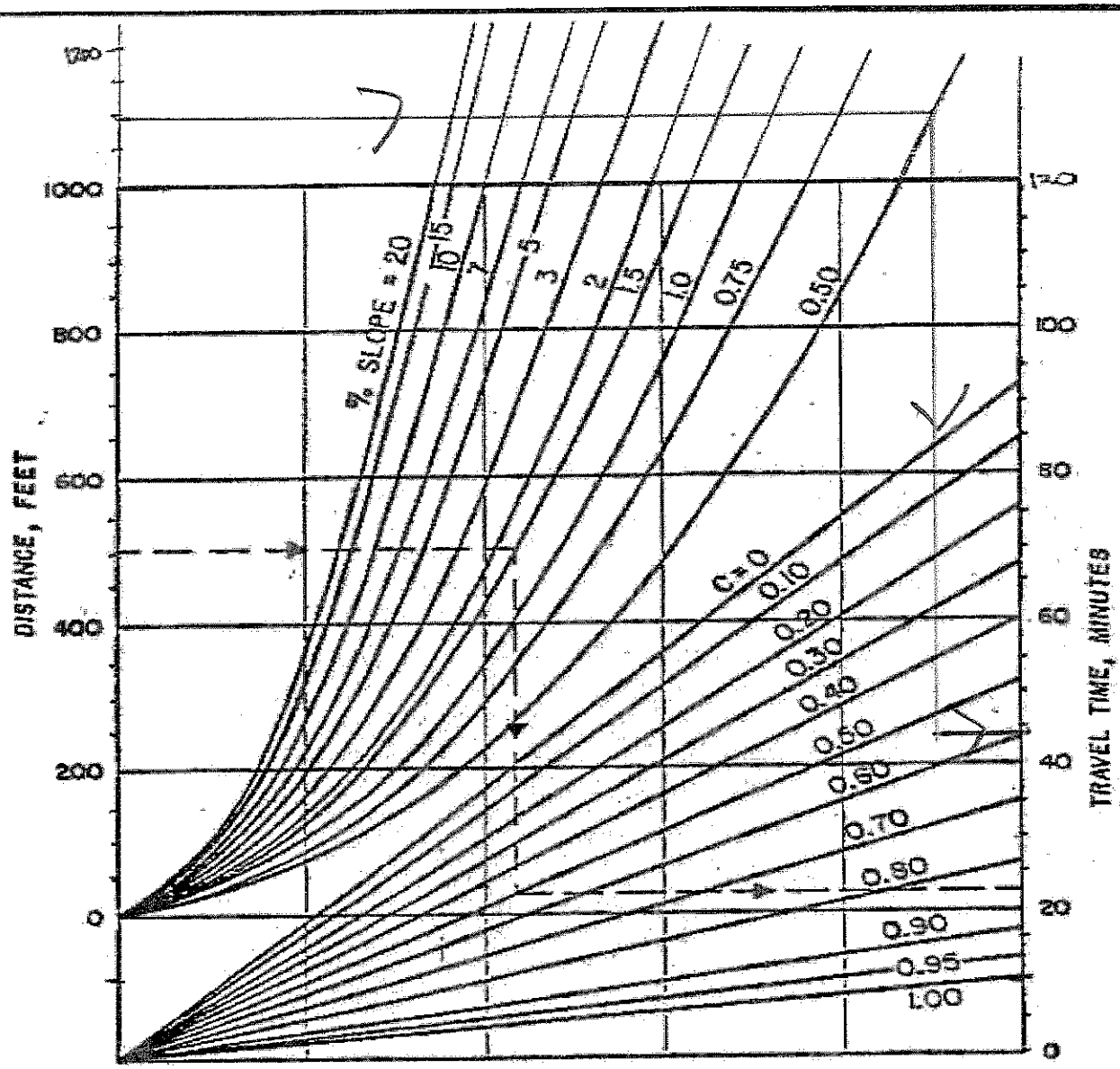
Curve - 0.5

Slope - 0.5%

Travel Time (T_c) = 48

Rainfall Intensity = 0.99
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D1 (25 yr.)

Length - 1,089 ft

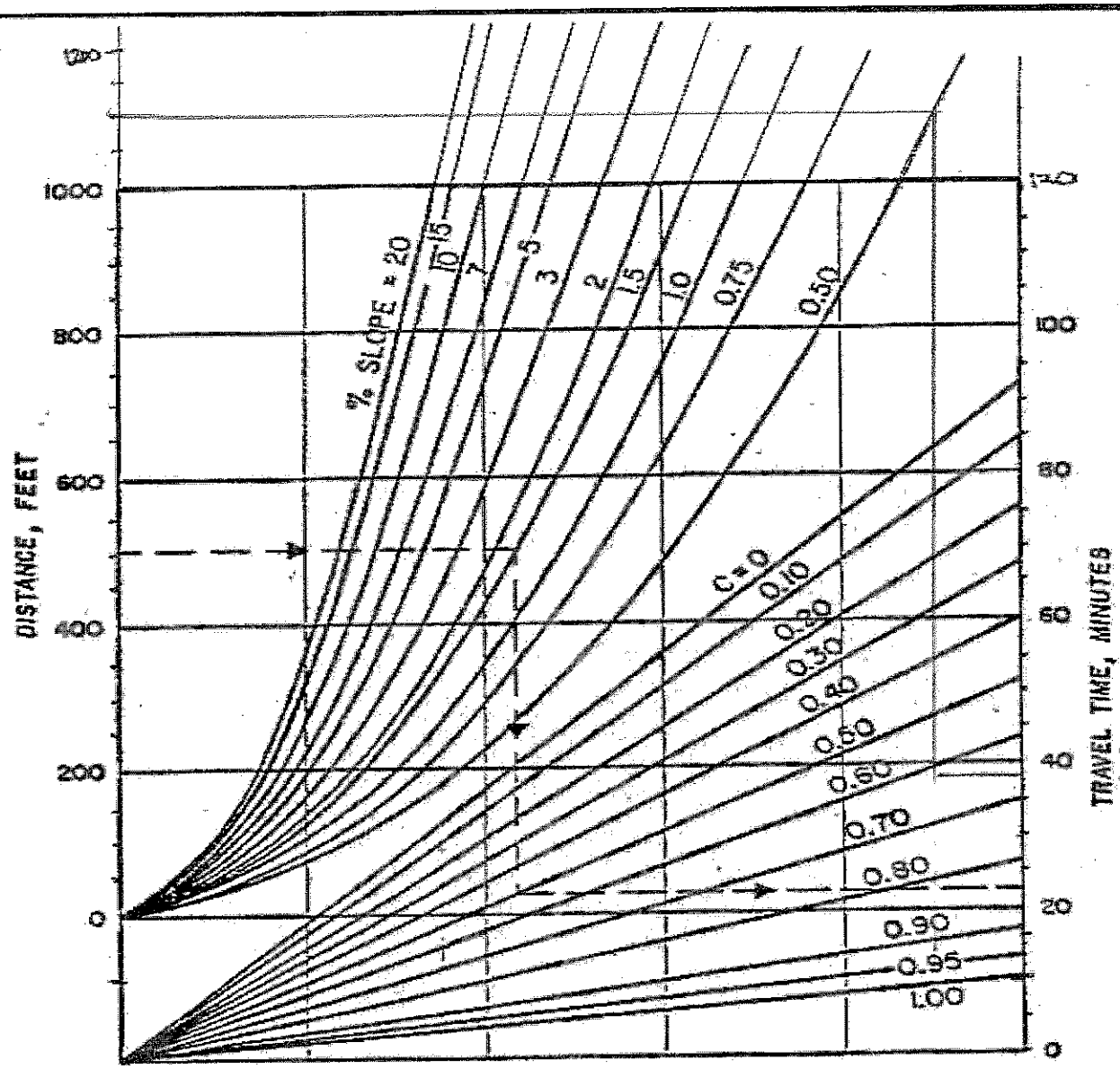
Curve - 0.59

Slope - 0.5%

Travel Time (T_c) = 44

Rainfall Intensity = 1.24
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D1 (100 yr.)

Length - 1,089 ft.

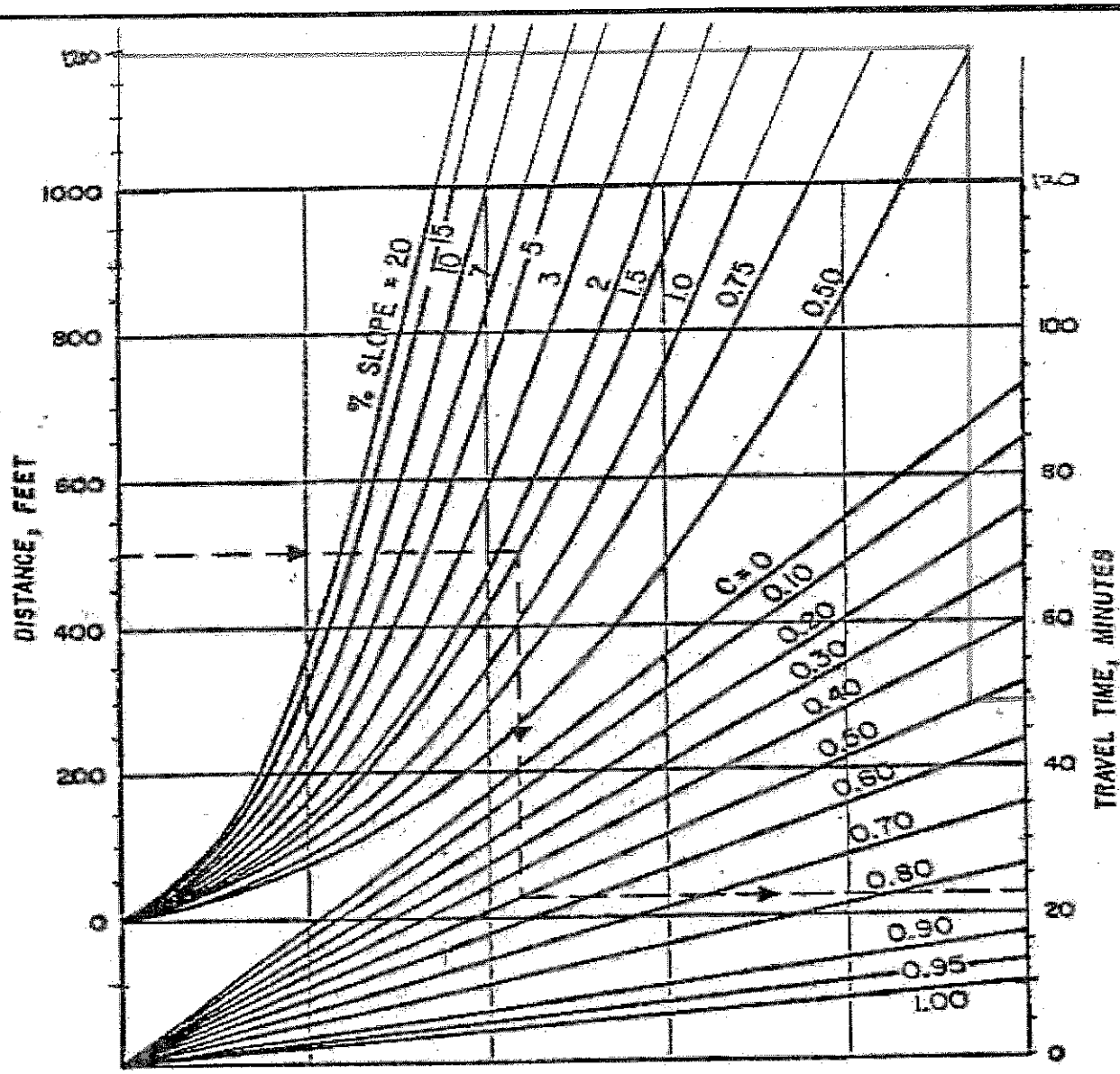
C value - 0.63

Slope - 0.5%

Travel Time (T_c) = 38

Rainfall Intensity = 1.76
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D2 (15 yr.)

Length - 1184

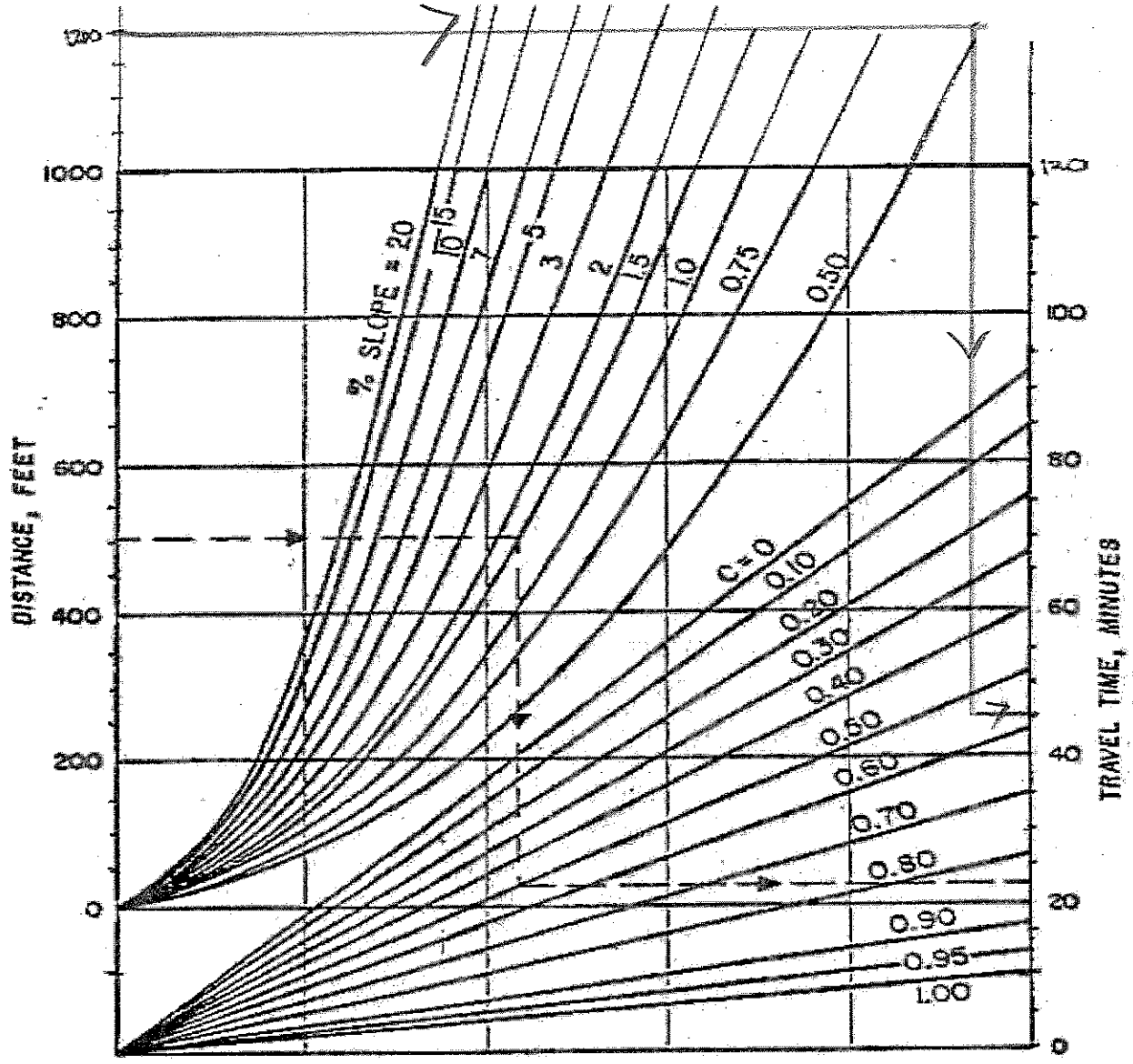
Cruke - 0.5

Slope - 0.5%

Travel Time (T_c) = 49

Rainfall Intensity = 0.97
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D2 (25-yr.)

Length - 1184 ft.

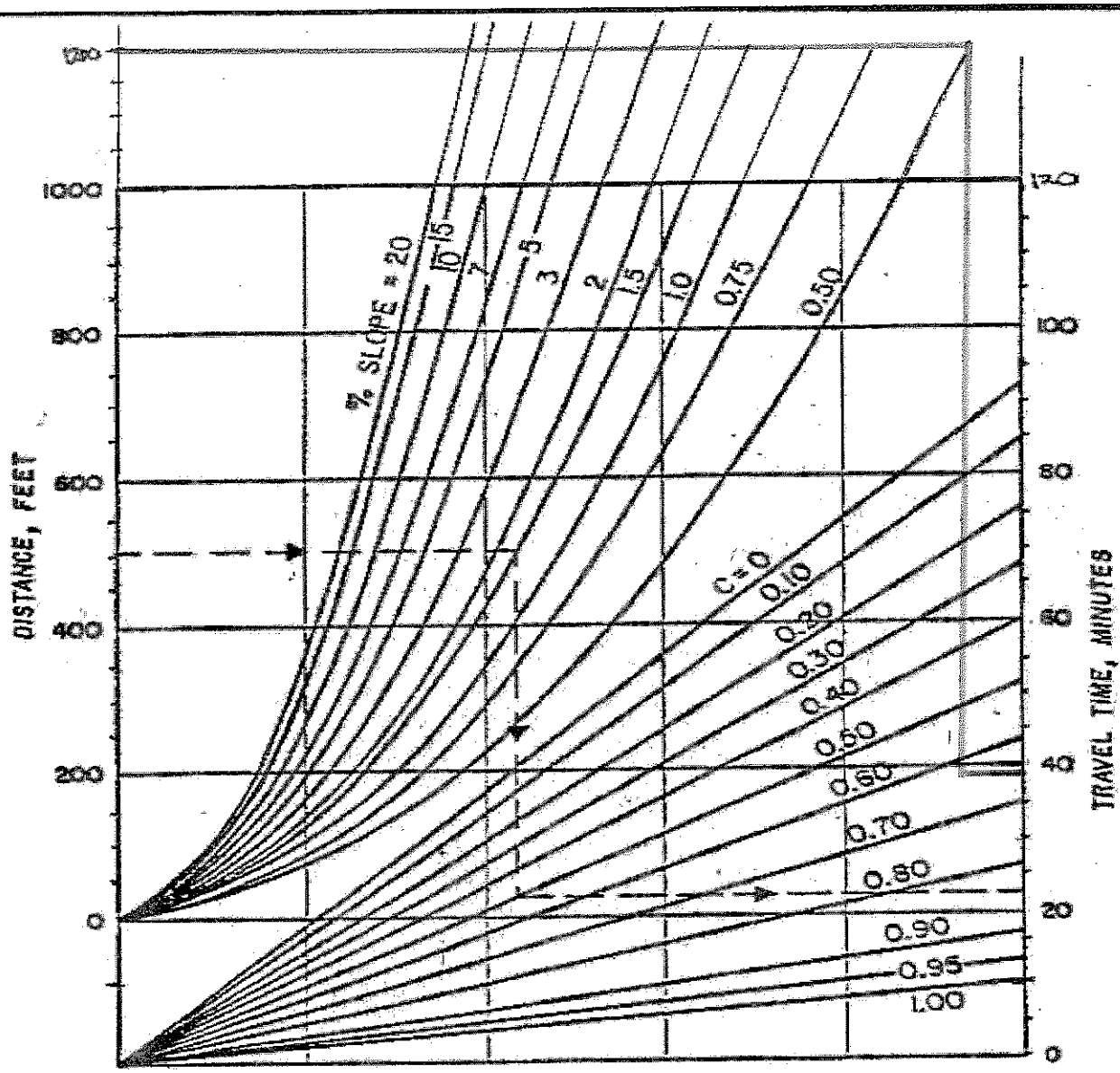
C value - 0.54

Slope - 0.5%

Travel Time (T_c) = 45

Rainfall Intensity = 1.22
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D2 (100-yr)

Length - 1184 ft.

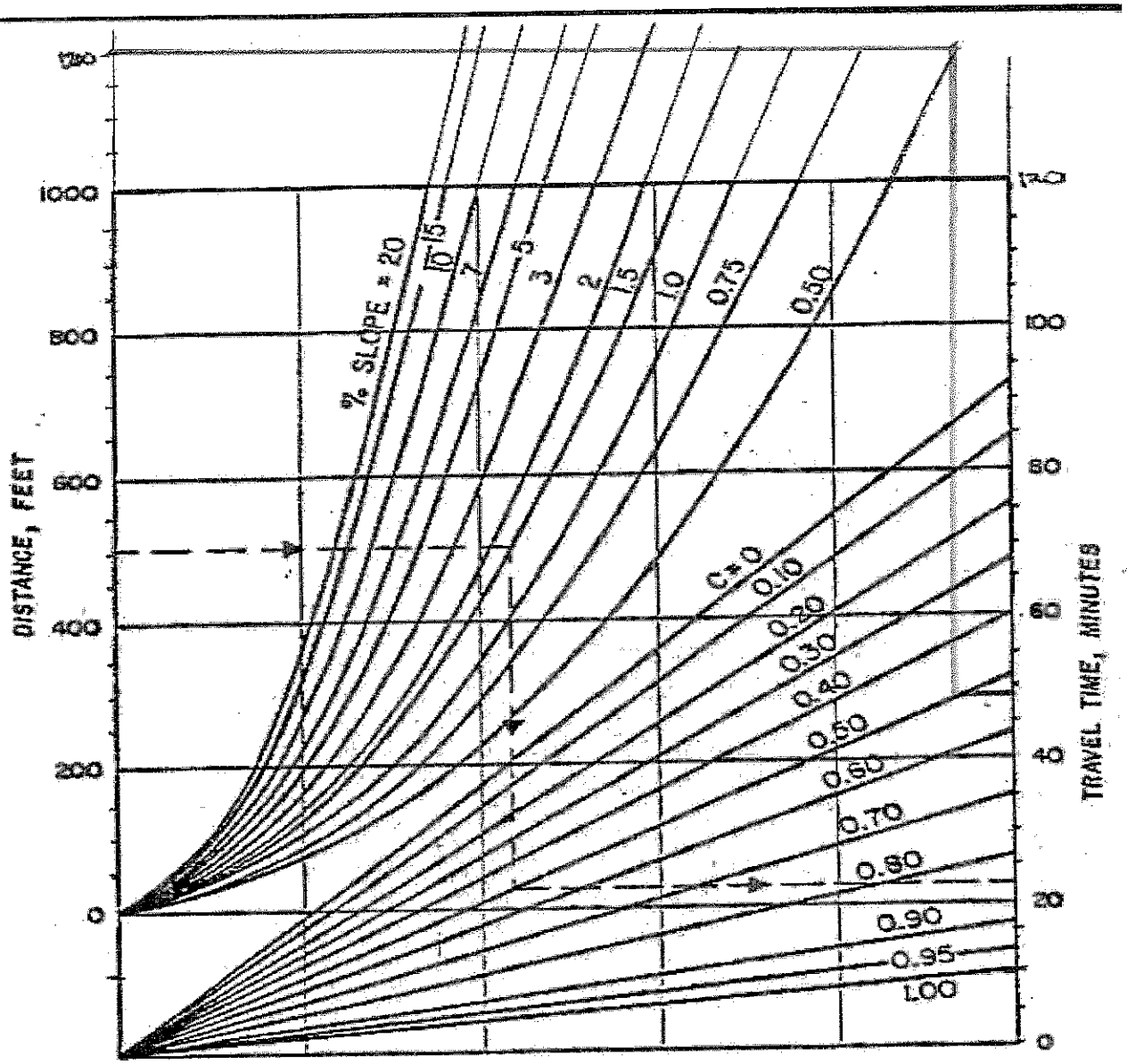
C value - 0.63

Slope - 0.5%

Travel Time (T_c) = 38

Rainfall Intensity = 1.76
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D3 (15 yr.)

Length - 1271 ft → 1200

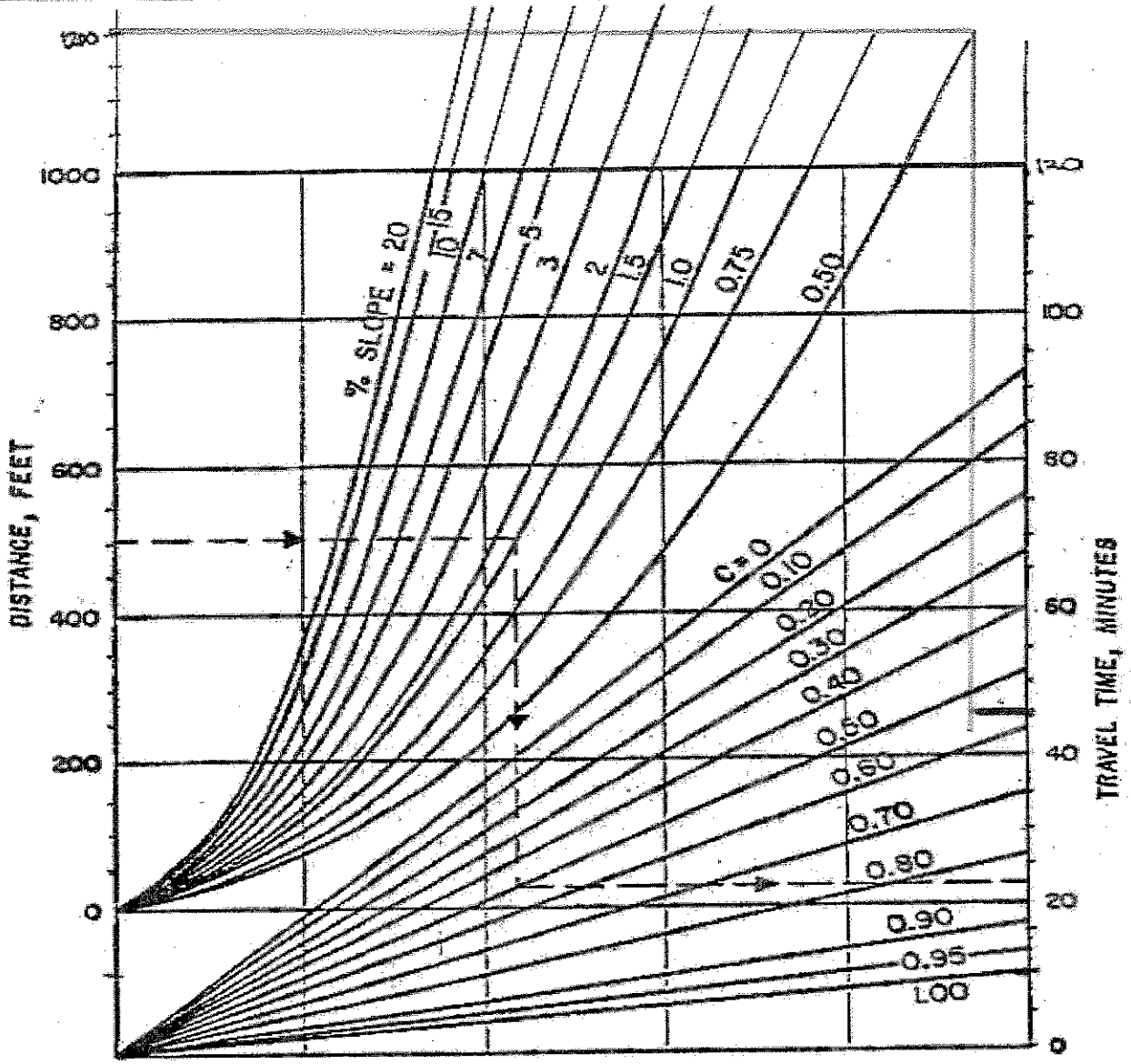
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 48

Rainfall Intensity = 0.99
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D3 (25 yr.)

Length - 1271 ft — 1200

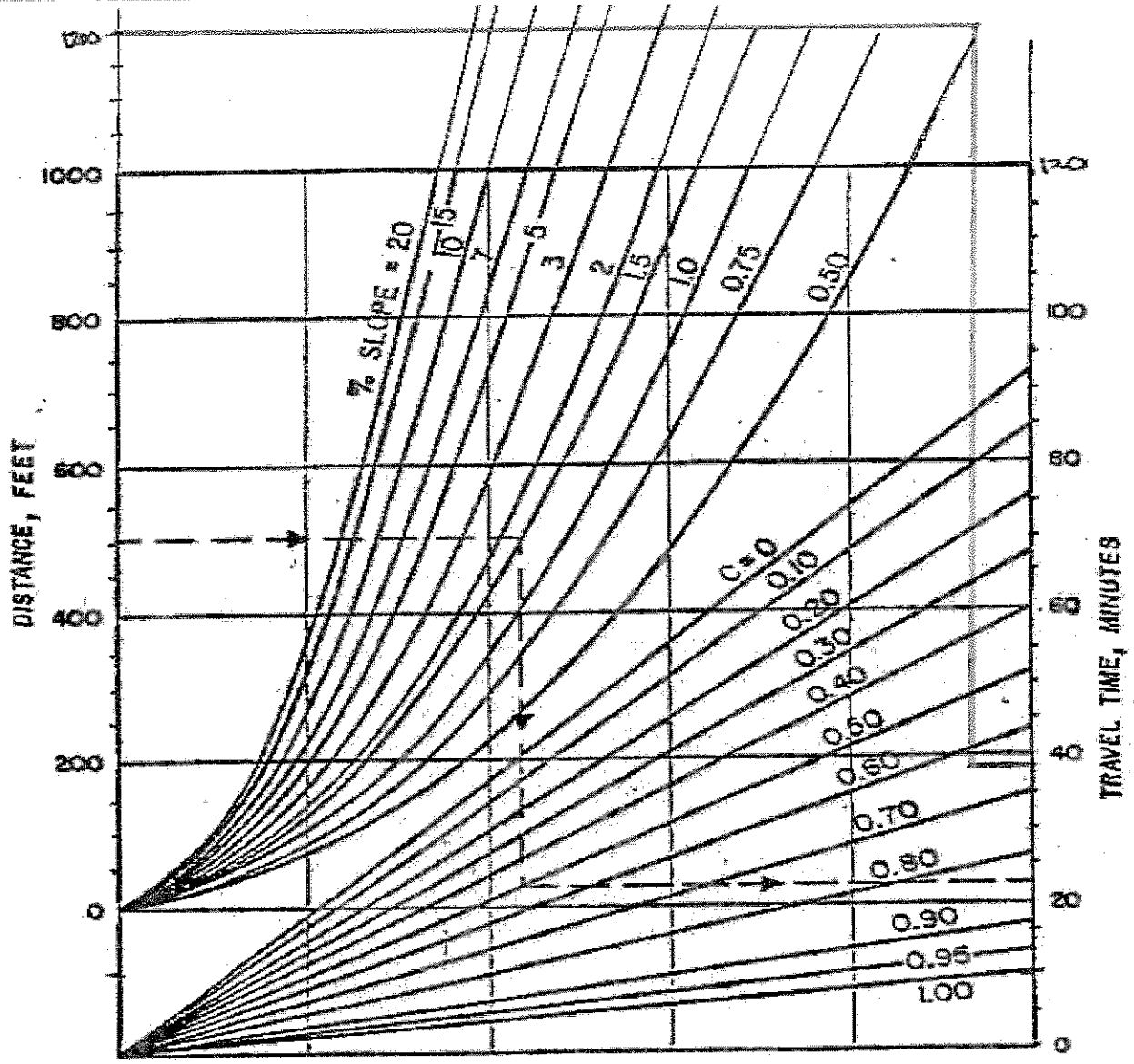
C value - 0.54

Slope - 0.5%

Travel Time (T_c) = 45

Rainfall Intensity = 1.22
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D3 (100 yr.)

Length - 1271 ft. — 1200

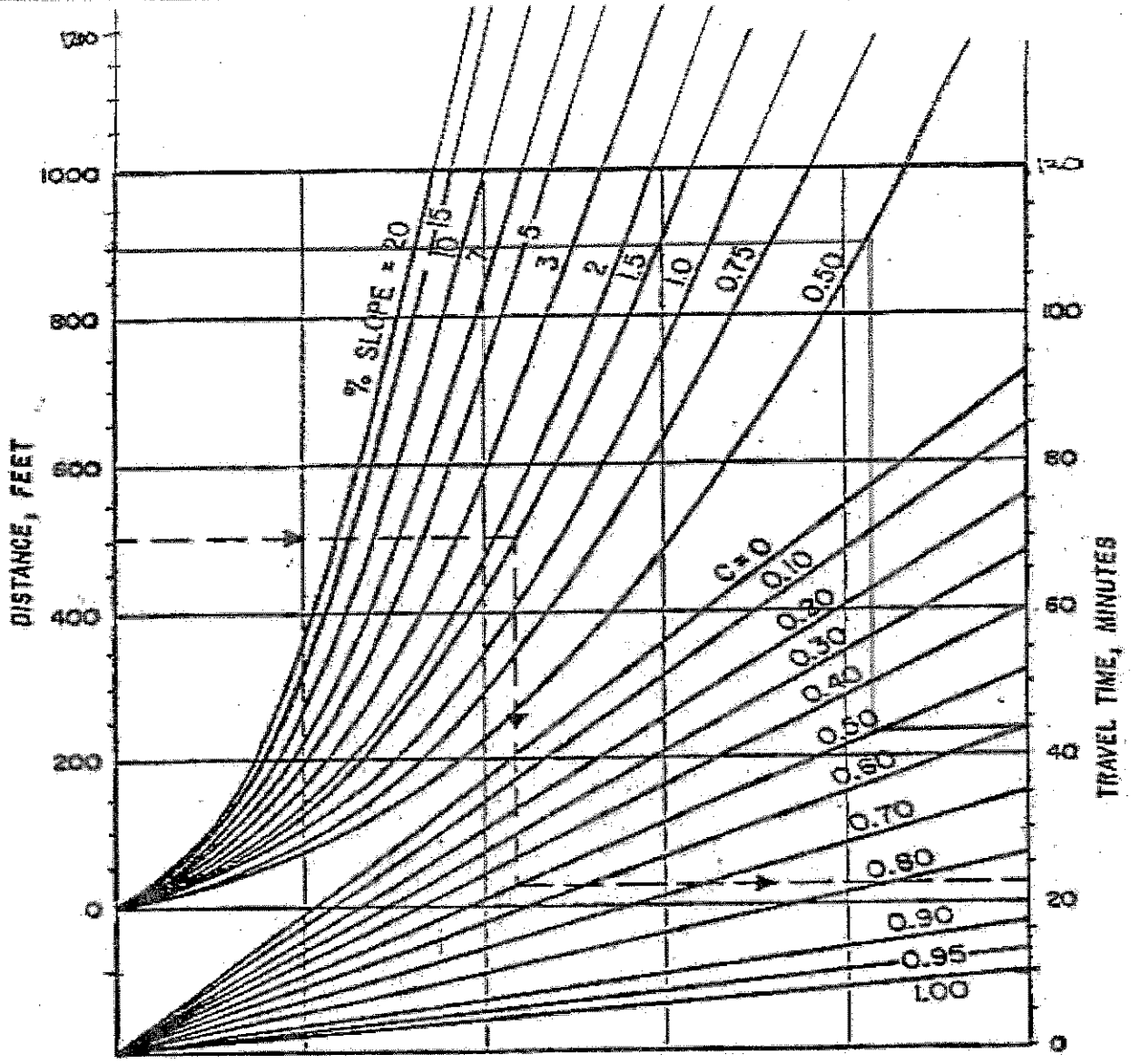
Curve - 0.63

Slope - 0.5%

Travel Time (T_c) = 38

Rainfall Intensity = 1.76
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D4 (15 yr.)

Length - 893 ft.

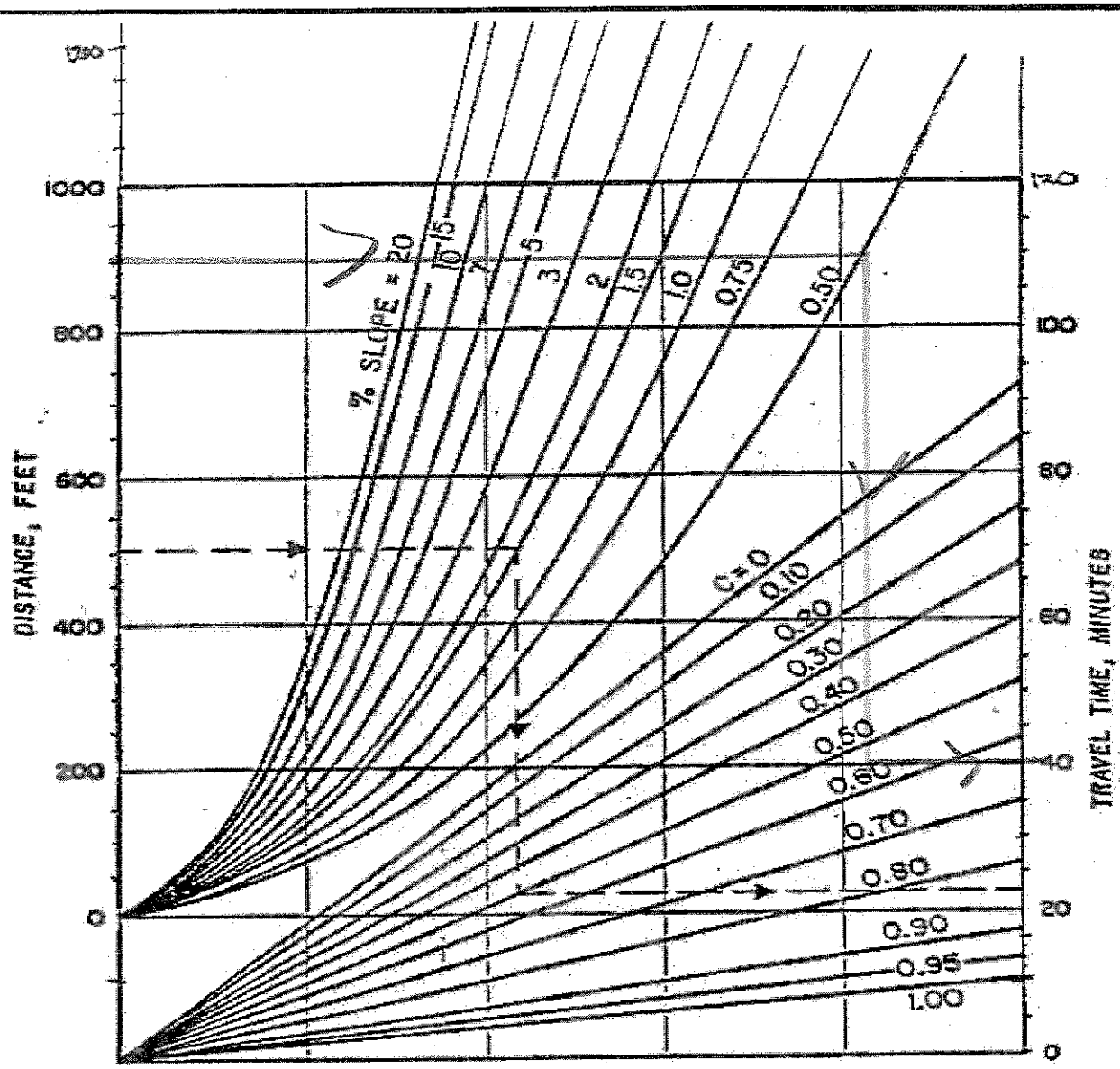
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 44

Rainfall Intensity = 1.03
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D4 (25 yr.)

Length - 893 ft.

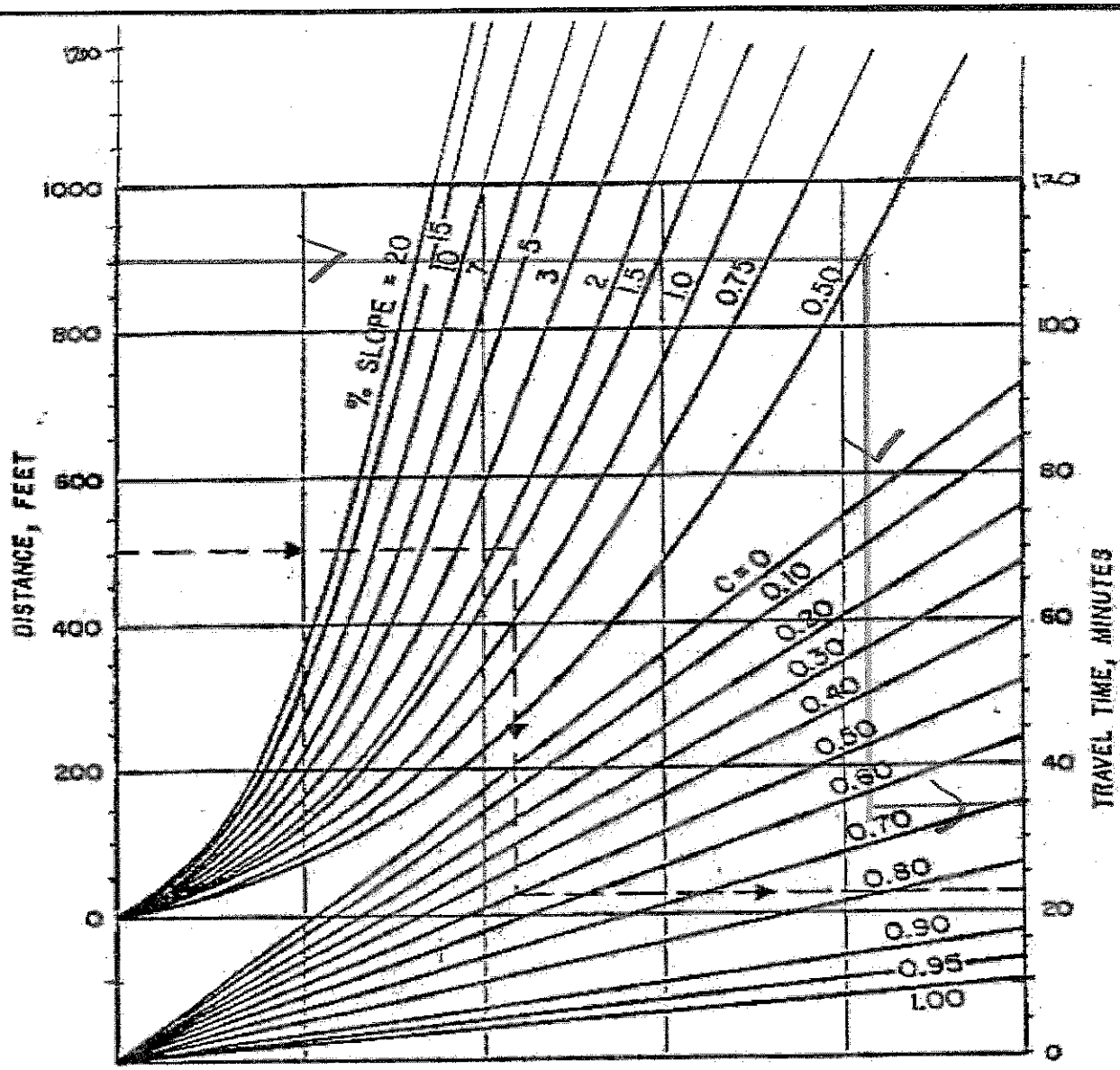
C value - 0.5

Slope - 0.5%

Travel Time (T_c) = 40

Rainfall Intensity = 1.32
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW



Basin D4 (100 yr.)

Length - 893 ft.

C value - 0.63

Slope - 0.5%

Travel Time (T_c) = 35

Rainfall Intensity = 1.85
(Table 2)

FIGURE 1.
TRAVEL TIME FOR OVERLAND FLOW

	Existing Condtions			W/ Project		
	15 year	*25 year*	*100 year*	*15 year*	*25 year*	*100 year*
Basin A1	4.19	5.62	9.13 *cfs*	4.19	5.62	9.13 *cfs*
Basin A2	7.67	11.12	14.59 *cfs*	7.67	11.12	14.59 *cfs*
Basin A3	7.57	10.15	16.33 *cfs*	27.76	43.51	77.79 *cfs*
Basin B	7.94	11.97	15.71 *cfs*	7.94	11.97	15.71 *cfs*
Basin C1	1.45	1.98	3.17 *cfs*	1.45	1.98	3.17 *cfs*
Basin C2	11.40	9.60	18.96 *cfs*	11.40	9.60	18.96 *cfs*
Basin D1	2.23	2.99	4.95 *cfs*	2.23	2.99	4.95 *cfs*
Basin D2	6.21	8.35	14.08 *cfs*	6.21	8.35	14.08 *cfs*
Basin D3	4.06	5.35	9.02 *cfs*	4.06	5.35	9.02 *cfs*
Basin D4	1.24	1.69	2.78 *cfs*	1.24	1.69	2.78 *cfs*
Total/Outlet	53.95	68.82	108.71 *cfs*	74.13	102.18	170.18 *cfs*
Net Difference	**20.19**	**33.36**	**61.46**			
Convergance SR12/Walters Road	40.21	50.43	77.89	60.40	83.79	139.35
net	**20.19**	**33.36**	**61.46**			

0.5% (Existing) $\frac{40.21}{0.07071} = 596.9$ $\frac{50.43}{0.07071} = 713.2$ $\frac{77.89}{0.07071} = 1101.5$

(w/ Project) $\frac{60.40}{0.07071} = 854.2$ $\frac{83.79}{0.07071} = 1185.0$ $\frac{139.35}{0.07071} = 1970.7$

1%

	Existing Condtions			W/ Project		
	15 year	*25 year*	*100 year*	*15 year*	*25 year*	*100 year*
Basin A1	4.90	6.42	10.42 *cfs*	4.90	6.42	10.42 *cfs*
Basin A2	8.37	8.54	13.30 *cfs*	8.37	8.54	13.30 *cfs*
Basin A3	8.57	11.31	18.72 *cfs*	31.47	45.40	86.94 *cfs*
Basin B	9.02	8.40	18.82 *cfs*	9.02	8.40	18.82 *cfs*
Basin C1	1.70	2.32	3.92 *cfs*	1.70	2.32	3.92 *cfs*
Basin C2	12.08	16.37	21.50 *cfs*	12.08	16.37	21.50 *cfs*
Basin D1	2.57	3.44	5.68 *cfs*	2.57	3.44	5.68 *cfs*
Basin D2	7.17	9.66	15.68 *cfs*	7.17	9.66	15.68 *cfs*
Basin D3	4.55	6.14	10.05 *cfs*	4.55	6.14	10.05 *cfs*
Basin D4	1.43	1.93	3.14 *cfs*	1.43	1.93	3.14 *cfs*
Total/Outlet	60.35	74.53	121.21 *cfs*	83.25	108.62	189.44 *cfs*
Net Difference	**22.90**	**34.10**	**68.22**			
Convergance SR12/Walters Road	44.64	53.36	86.67	67.54	87.46	154.90
net	**22.90**	**34.10**	**68.22**			

Capacity Determination

(Existing) 1% Slope $\frac{44.64}{0.1} = 446.4$ $\frac{53.36}{0.1} = 533.6$ $\frac{86.67}{0.1} = 866.7$

(w/ Project) $\frac{67.54}{0.1} = 675.4$ $\frac{87.46}{0.1} = 874.6$ $\frac{154.90}{0.1} = 1549.0$