CROSS-DRAINAGE & CULVERT DESIGN ON HIGHWAYS

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İZMİR

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We certify that we have read this thesis and that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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ABSTRACT

One of the most important considerations of the location and design of rural highways is the necessity for providing adequate drainage. Highway drainage may be generally defined as the process of controlling and taking away excess surface and subsurface water from the road structure in order to prevent any damage on it. Cross drainage, on the other hand, involves the conveyance of surface water and stream flow across or from the highway right of way.

Highway culverts are used to transport water under a highway to allow water to continue to a natural water source (e.g., a river or lake). Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Many different inlet and outlet configurations are used on culvert barrels to reduce erosion, inhibit seepage, retain the fill, improve the aesthetics and hydraulic characteristics and make the ends structurally stable. Futhermore, in order for an efficient and economic operation of a culvert within a drainage system, culverts should be located properly and the required length for a culvert should be selected carefully.

Regardless of the size or cost of the drainage feature, the most important step prior to hydraulic design is estimating the discharge (rate of flow) or volume of runoff that the drainage facility will be required to convey or control, and cross drainage design normally requires more extensive hydrologic analysis than is necessary for roadway drainage design. Having estimated the design discharge to be conveyed, the next step is the hydraulic design of a culvert that consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other.

In this thesis work, hydrologic and hydraulic considerations for culverts are mentioned together with some general features related to culverts and an alternative culvert design procedure compiled from national and international literature is proposed based on all these mentioned criteria. In addition, to illustrate the real phenomenon, certain culvert locations throughout Aydın-Denizli Highway are selected and re-designed both by hand calculation and by using three related softwares, and the results obtained are compared in the end.

ÖZET

Kentdışı otoyolların tasarımında gözönünde bulundurulan faktörlerden en önemlisi yol için yeterli drenajın sağlanması zorunluluğudur. Karayolu drenajı, genel olarak gerek yeraltındaki gerekse yüzeydeki fazla suyun yol yapısına zarar vermesinin önlenebilmesi için kontrollu bir şekilde yoldan uzaklaştırılması şeklinde tanımlanabilir. Enine drenaj ise, yüzey suları ve yolu kesen nehir sularının yolun diğer tarafına iletilmesini ifade eder.

Karayolu menfezleri, suyun akışını sürdürerek doğal su kaynağına (nehir veya göl gibi) ulaşabilmesi için yol altından iletimini sağlar. Genellikle küçük su yapıları olarak düşünülmesine rağmen menfezlerin, yeterli drenajın ve karayolu bütünlüğünün sağlanmasında rolleri büyüktür. Erozyonun azaltılması, sızmanın önlenmesi, dolgunun korunması, estetiğin ve hidrolik kapasitenin yükseltilmesi ve giriş-çıkışların yapısal dayanımının artırılması için menfezlerde çeşitli giriş ve çıkış konfigürasyonları kullanılır. Bundan başka, menfezin, drenaj sistemi içinde verimli ve ekonomik işletiminin sağlanması için, yer ve uzunluk seçimine oldukça dikkat edilmesi gereklidir.

Drenaj yapısının büyüklüğü ve maliyetinden bağımsız olarak, menfez hidrolik tasarımından önceki en önemli aşama drenaj yapısının ileteceği debinin veya akış hacminin belirlenebilmesidir ve enine drenaj tasarımı, normal bir yol drenajından farklı olarak, daha kapsamlı bir hidrolojik analiz gerektirmektedir. İletilecek proje debisinin belirlenmesinden sonraki aşama da akışın, yolun bir tarafından diğer tarafına taşınmasında menfez performansının incelenmesini kapsayan hidrolik tasarım aşamasıdır.

Bu tez çalışmasında, menfezlerin bazı genel özellikleri ile birlikte menfezlere ilişkin hidrolojik ve hidrolik özellikler ifade edilmiş ve tüm bu bilgiler ışığında yerli ve yabancı kaynaklardan derlenerek elde edilen bir tasarım yöntemi alternatif olarak önerilmiştir. Ek olarak, gerçek olayı gösterme açısından, Aydın-Denizli Otoyolu üzerinde birkaç menfez yeri seçilmiş ve menfezler, hem el ile hesap yöntemiyle hem de menfez tasarımına yönelik üç yazılımın kullanılmasıyla yeniden tasarlanarak sonuçlar değerlendirilmiştir.

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CHAPTER ONE
INTRODUCTION

1.1 Highway Drainage

One of the most important considerations of the location and design of rural highways is the necessity for providing adequate drainage. Adequate and economical drainage is essential for the protection of the investments made in a highway structure and for the safety of the people who use it.

Highway drainage may be generally defined as the process of controlling and taking away excess surface and subsurface water from the road structure in order to prevent any damage on it. Correspondingly, road drainage considerations can conveniently be divided into those relating to the flow of surface water and of subsurface water. Subsurface drainage is concerned with the flow of water within soils, while surface drainage is concerned with the measures taken to control the movement of water over the ground.

The negative effect of water on a highway structure can be summarized in four groups:

a) Water may increase the plasticity of soil on which the embankment and the road exist and may lead to a decrease in the bearing capacity of soil.

b) Water may cause sliding of the soil layers forming side slopes or foundation of the road, by producing a shear surface between them.

c) As the water percolating down into the road fill would freeze in cold weather and would thaw in spring, the presence of water within the road structure may cause deterioration of the road under traffic action.

d) Erosion caused by water may also deteriorate surface course, shoulders, cut and fill slopes within the road structure.

1.2 Objectives of Drainage Design

Drainage design is mainly based on the necessity for preventing the retention of water by the highway and providing the removal of water from the roadway considering all related factors.

Basic steps to be taken for the above purpose generally include: (Highway Design Manual, 1995, p. 800-2)

(a) Estimating the amount and frequency of storm runoff.

(b) Determining the natural points of concentration and discharge, the limiting elevations of entrance head, and other hydraulic controls.

(c) Estimating the amount and composition of bedload and its abrasive and bulking effects.

(d) Determining the necessity for protection from floating trash and from debris moving under water.

(e) Determining the requirements for energy dissipation and bank protections.

(f) Determining the necessity of providing for the passage of fish and recognizing other ecological conditions and constraints.

(g) Analyzing the deleterious effects of corrosive soils and waters on structures.

(h) Comparing and coordinating proposed design with existing drainage structures and systems handling the same flows.

(i) Coordinating, with local agencies, proposed designs for facilities on roads to be relinquished.

(j) Providing access for maintenance operations.

(k) Providing the removal of detrimental amounts of subsurface water.

(l) Designing the most efficient drainage facilities consistent with the factors listed above, economic considerations, the importance of the highway, and

(m) Checking the structural adequacy of designs.

1.3 Surface Drainage

As mentioned above, surface drainage is related to the removal of water over the ground and to corresponding measures taken to control the movement of this water. It is convenient to divide a surface drainage study into two parts. The first consideration is related to the problem of deciding the amount of water arriving at the drainage ditch or culvert, and the second one is related to the design of the facility needed to handle this amount of water. The first part of a complete design work is termed the hydrological design, while the latter is called the hydraulic design.

CHAPTER TWO
CROSS-DRAINAGE

2.1 Cross-Drainage

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some other type of flow obstruction.

2.2 Culverts
2.2.1 Definition

Highway culverts are used to transport water under a highway to allow water to continue to a natural water source (e.g., a river or lake). In other words, the function of a culvert is to convey surface water across or from the highway right-of-way. Since a bridge performs the same function, there may be a confusion between a bridge and a culvert. Usually, a structure that is 20 feet (~ 6 m) or less in span length is considered to be a culvert, and structures greater than 20 feet in span length are considered to be a bridge (Ritter & Paquette, 1978). However, this line of division is not standard and different span lengths are employed by various organizations as limiting culvert lengths.In Turkey, the limiting value is generally assumed to be 10 m to distiguish between a culvert and a bridge (Yıldırım, 1984). Another distinction between them is that a bridge surface forms a part of the roadway surface whereas the top of a culvert does not ordinarily form a part of the carriageway. One futher difference between them is that culverts may very often be designed to flow full, whereas bridges are normally designed to pass floating debris and, possibly, boats.

Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is relatively small, the cumulative cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the cost of maintaining highway drainage features is substantial, and culvert maintenance is a large share of these costs. Improved service to the public and a reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

Culverts are located in three general locations: at the bottom of depressions where no natural watercourse exists; at locations where natural streams intersect the roadway; and at locations required for passing surface drainage carried in side ditches beneath roads to adjacent territory.

2.2.2 Basic Recommendations For A Good Culvert

The following are some basic rules that should be taken into consideration in culvert design:

1) The culvert, appurtenant entrance and outlet structures should properly take care of water, bed load, and floating debris at all stages of flow.
2) It should cause no unnecessary or excessive property damage.
3) Normally, it should provide for transportation of material without detrimental change in flow pattern above and below the structure.
4) It should be designed so that future channel and highway improvement can be made without too much loss or difficulty.
5) It should be designed to function properly after fill has caused settlement.
6) It should be designed to accommodate increased runoff caused by anticipated land development.
7) It should be economical to build, hydraulically adequate to handle design discharge, structurally durable and easy to maintain.

8) It should be designed to avoid excessive ponding at the entrance which may cause property damage, accumulation of sediment, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.
9) The outlet should be designed to resist undermining and washout.

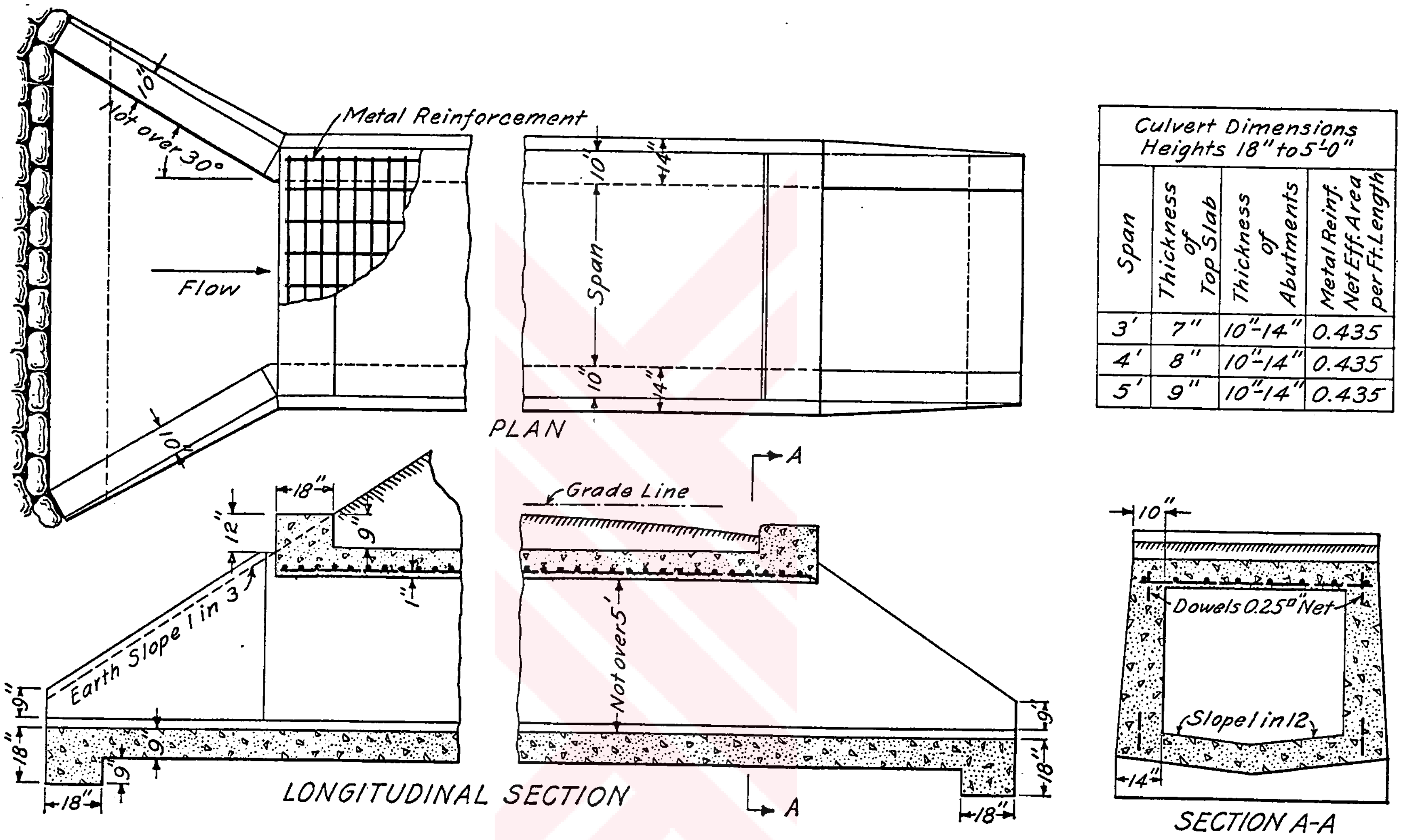
2.3 Culvert Types

Culvert type selection includes the choice of material, shape and cross section and the number of culvert barrels. The shape of a culvert is not the most important consideration at most sites. The basic aim is to obtain a high hydraulic performance. Rectangular, arch or circular shapes of equal hydraulic capacity are generally satisfactory. However, it is often necessary for the culvert to have a low cross sectional area because of the terrain or because of the limited fill height. Several commonly used culvert types are as follows:

2.3.1 Box or Rectangular Culvert Types

Rectangular or, with their widely used names, box culverts have certain standard types commonly used in highways. These standard culvert types have span lengths of 1.0, 1.5, 2.0, 2.5, 3.0 m and heights of 0.6, 1.0, 1.5, 2.0, 2.5, 3.0 m (Atalay et al., 1973). Whenever needed, it is possible to place multiple barrels of box culvert type (with two or three opening) in wide channels where the concentration of flow is to be kept to a minimum. Moreover, a rectangular culvert lends itself more readily than other shapes to low allowable headwater situations, since the height may be decreased and the total span increased to satisfy the location requirement(Figure 2.1).

According to the statical design cosiderations, the maximum heights of the fill to be carried by the culvert are readily defined. Culverts of box type having span lengths between 1.0 and 1.5 m can be applied under a maximum of 15 m fill, while the ones having span lengths of 2.0, 2.5 and 3.0 m can be used under a maximum of 9 m fill. Besides, box culverts consisting of two or three cells can be used under a 6m fill (Atalay et al., 1973) (Figure 2.2 & 2.3).



Culvert Dimensions Heights 18" to 5'-0"			
Span	Thickness of Top Slab	Thickness of Abutments	Metal Reinf. Net Eff. Area per Ft. Length
3'	7"	10"-14"	0.435
4'	8"	10"-14"	0.435
5'	9"	10"-14"	0.435

Figure 2.1 Design for medium-sized reinforced concrete box culvert (Bruce&Clarkeson, 1960)

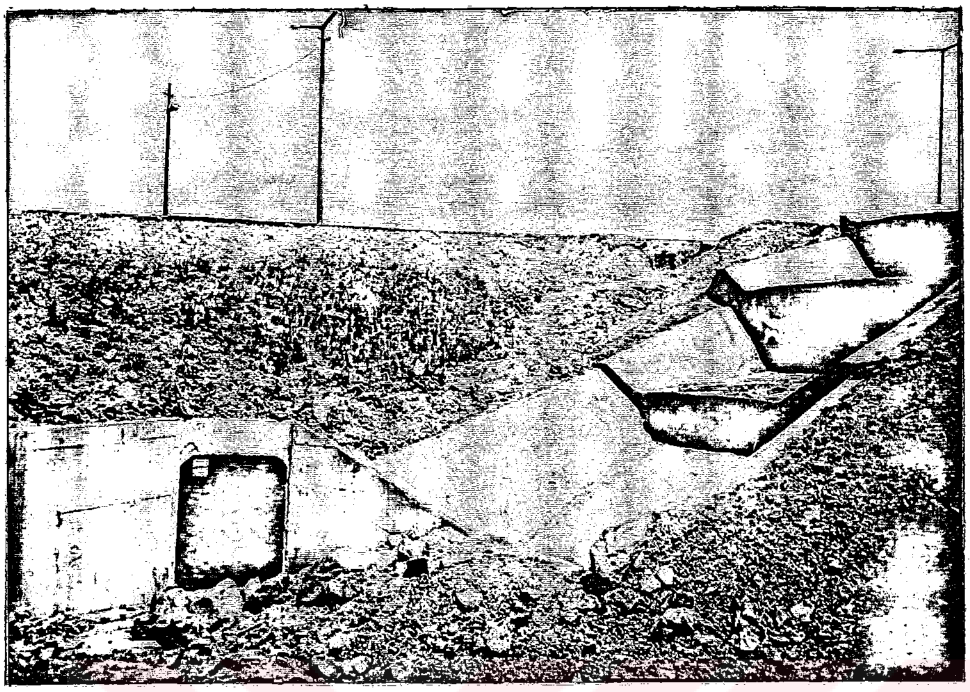


Figure 2.2 Upstream inlet of a box culvert under the embankment

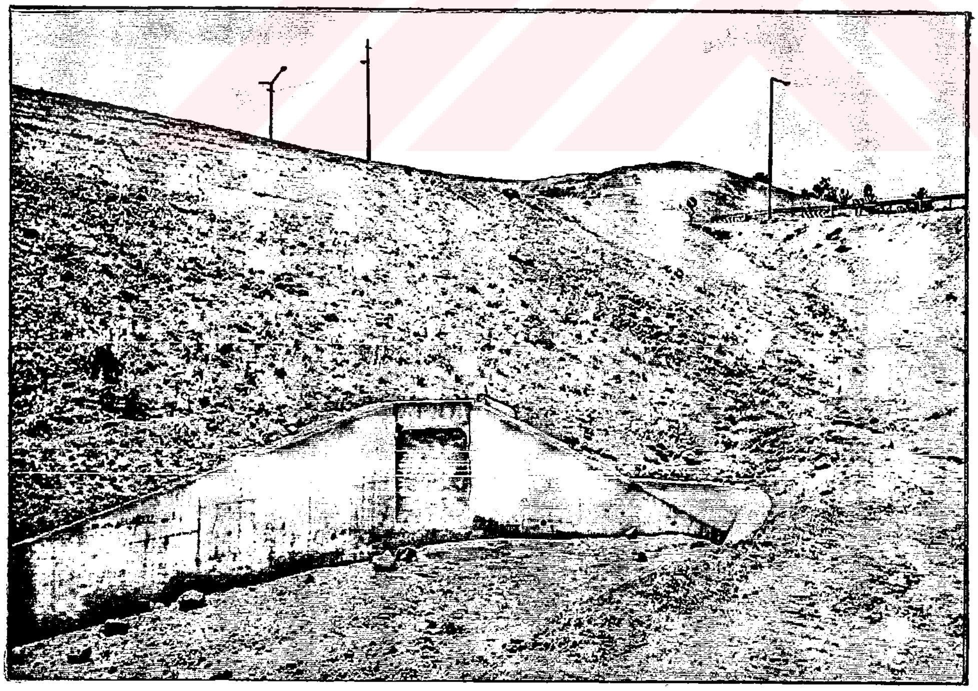


Figure 2.3 Downstream outlet of a box culvert under the embankment

Box culverts, the maximum allowable grade of which is 0.15 (Atalay et al., 1973), can be commonly used on foundations having low bearing capacity. However, this type of culverts should not be used on streams where the sediment transport and the velocities are high.

2.3.2 Circular Culvert Types

The most commonly used culvert shape is circular. This shape is structurally efficient under most loading conditions. Various standard lengths of circular pipe in standard strength classes are usually available from local suppliers. Standard circular culverts have three general types with diameters of 0.8, 1.0 and 1.2 m (Atalay et al., 1973). Circular culverts are generally used in locations where the fill over the culvert is not sufficiently high. In other words, they should not be utilized under very high fills, in locations where sediment transport is high or where scouring by water is a major problem. Circular culverts are generally prefabricated, they can be used up to a 0.15 grade and the height of fill over circular culverts should not be greater than 0.7 m.

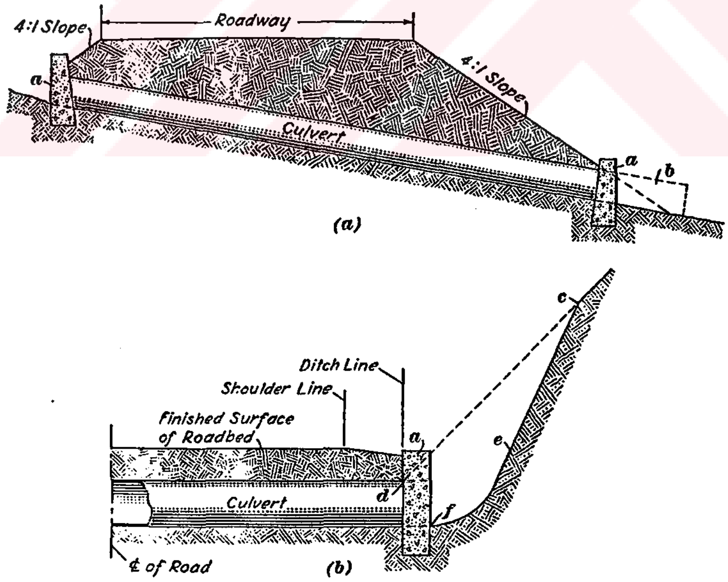


Figure 2.4 Proper installation of pipe culverts (a) in fill and (b) in cut (Bruce&Clarkeson, 1960)

In general, concrete or masonry headwalls are constructed in upstream end of circular culverts and downstream end bottom of the culvert is covered with big stones in locations where scouring occurs. As the foundation of circular culvert should be on strong or rocky soils, culvert is generally placed on a compacted soil in cases where the soil is weak, and then culvert is fixed by laying and compacting 15 cm soil layers around it (Figure 2.4 & 2.5).

2.3.3 Arch Culvert Types

This type of culverts having a variety of span lengths (0.7, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0 m) and various heights (0.6, 1.0, 1.5 m) have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Such structures can be installed to maintain the natural stream bottom for fish passage, but the potential for failure from scour must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations but have the advantage of rapid construction and low transportation and handling costs compared to the concrete and masonry ones. Arch culverts cannot be used in foundations where the bearing capacity of soil is below 2 kg/cm^2 (Aşkan, 1996) (Figure 2.6).

2.3.4 Bridge Culvert Types

Culverts the distance between whose supports is less than 10 m and the span length of which does not exceed 10 m are known as bridge culverts. As it is known structures having span lengths greater than 10 m are called bridges.

Bridge culverts include concrete or masonry side supports, wingwalls and the floor. The floor of such culverts having span length up to 6.5 m is constructed as slab, while the floor of ones having span length greater than that is constructed by using beams.

Such culverts cannot be used under fills. They are generally placed on locations where sediment transport, or water amount reaching the culvert, or span length is high or on locations where the height of fill does not allow any other type of culvert construction (Figure 2.7).

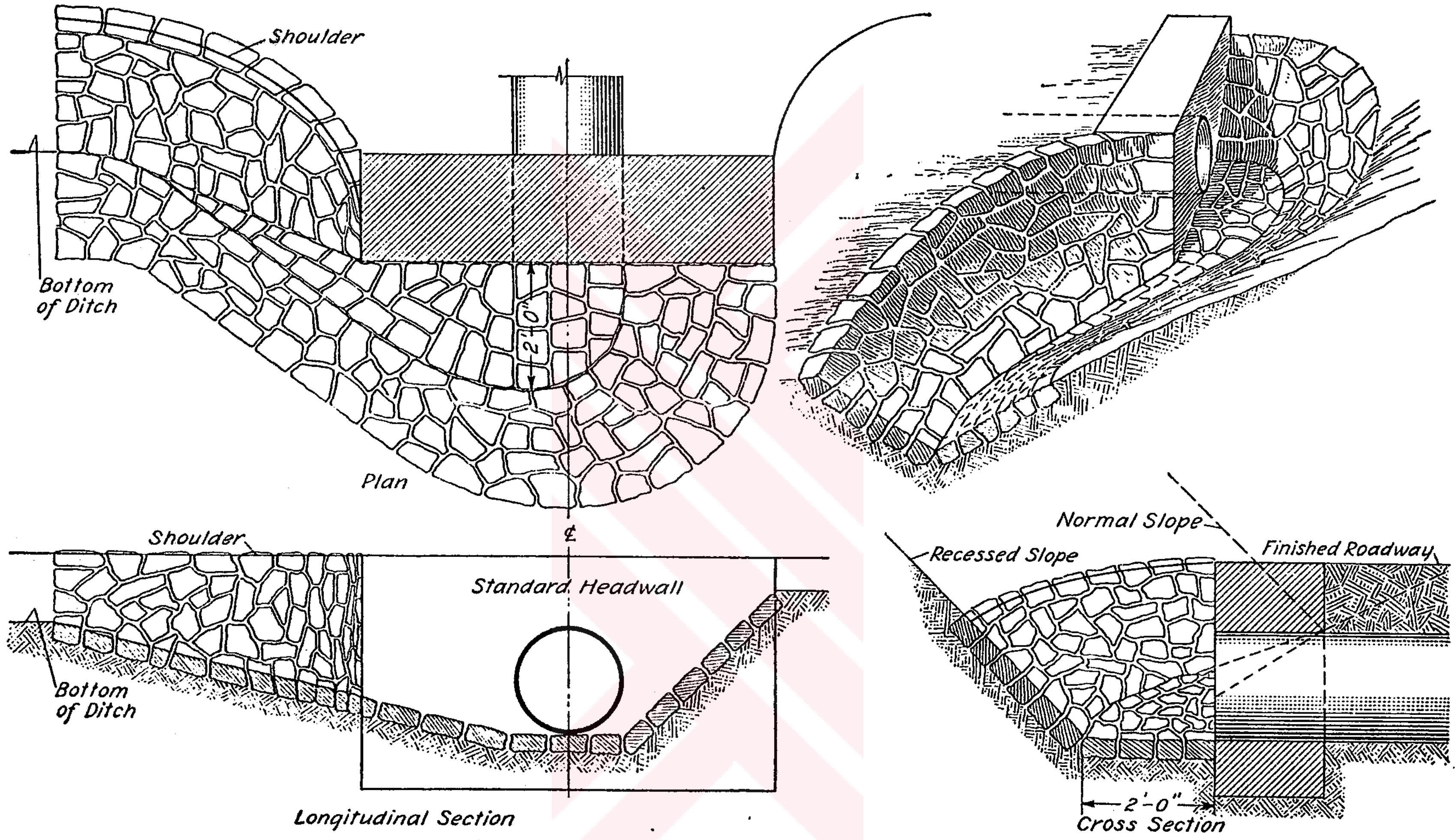
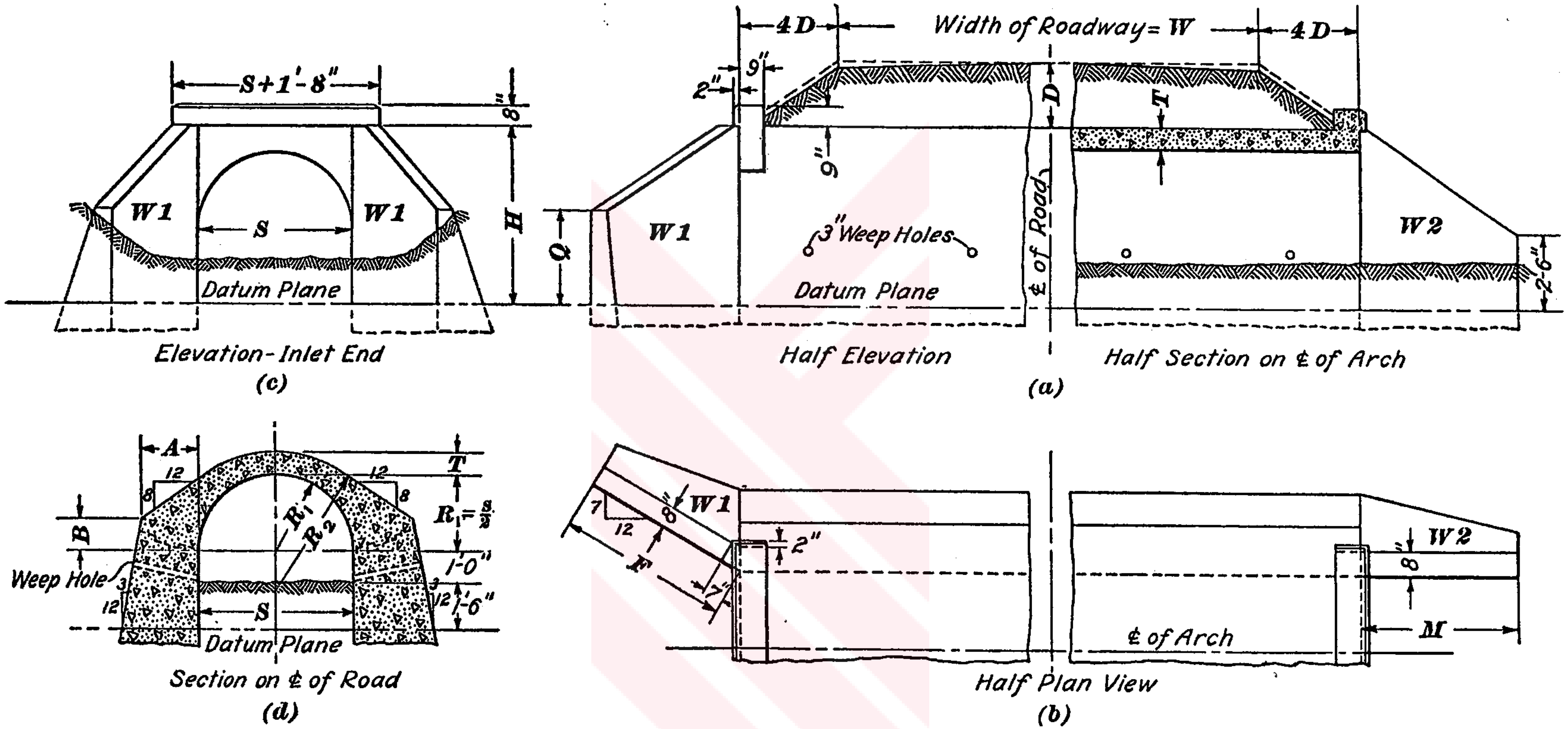


Figure 2.5 Design of a paved inlet to a culvert pipe (Bruce&Clarkeson, 1960)



S	Waterway	R_1	R_2	T	A	B	F	Q	H	M
2	3.6 sq ft	1'–0"	1'–6"	6"	8"	8"	2'–6"	3'–0"	4'–1"	2'–6"
3	6.5 sq ft	1'–6"	2'–3"	6"	1'– 1"	9"	3'–3"	3'–0"	4'–7"	3'–3"
4	10.3 sq ft	2'–0"	3'–0"	7"	1'– 5"	11"	4'–0"	3'–3"	5'–2"	4'–2"
5	14.8 sq ft	2'–6"	3'–9"	7½"	1'–10"	1'–0"	4'–9"	3'–3"	5'–8½"	5'–0"
6	20.1 sq ft	3'–0"	4'–6"	8"	2'– 1½"	1'–2"	5'–6"	3'–6"	6'–3"	5'–9"
7	26.2 sq ft	3'–6"	5'–3"	8½"	2'– 4"	1'–5"	6'–3"	3'–6"	6'–9½"	6'–7"

Figure 2.6 Design for small concrete arch culvert (Bruce&Clarkeson, 1960)

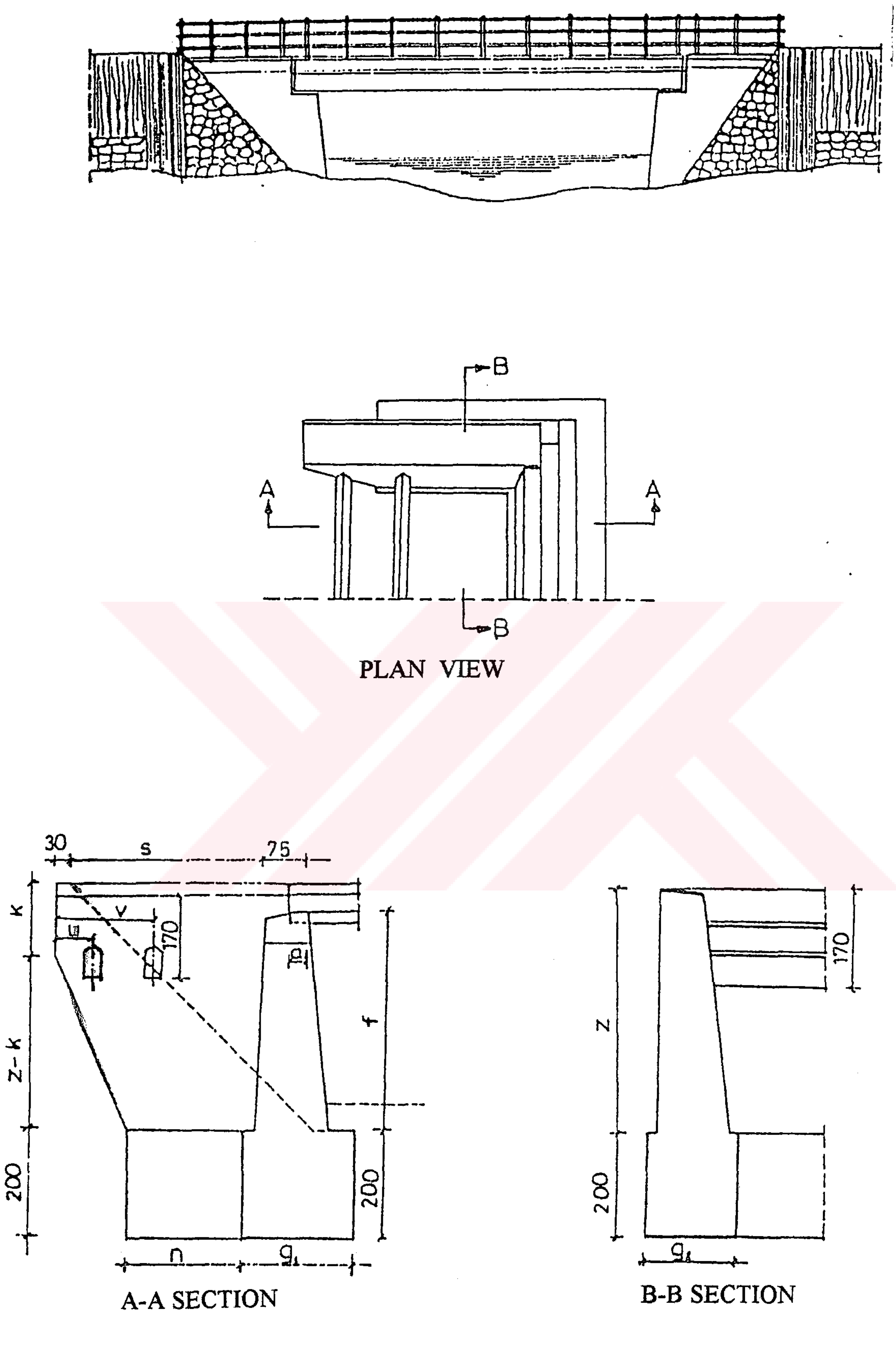


Figure 2.7 General and cross-sectional view of a bridge culvert (Aşkan, 1996)

2.4 End Treatments

Many different inlet and outlet configurations are used on culvert barrels. They can either be prefabricated or costructed in place, and common end configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Culvert end structures are attached to the ends of a culvert barrel to reduce erosion, inhibit seepage, retain the fill, improve the aesthetics and hydraulic characteristics and make the ends structurally stable.

A culvert is considered to have a projecting inlet or outlet when the culvert barrel extends beyond the face of the roadway embankment. This common type of culvert end has no end treatment and is vulnerable to various types of failures. The projecting end is economical but its appearance is not pleasing and its use should be limited to smaller culverts placed at minor locations.

A mitered culvert end is formed when the culvert barrel is cut to conform with the plane of the embankment slope. This type of treatment is used primarily with large metal culverts to improve the aesthetics of the culvert ends. It is structurally inadequate to withstand hydraulic, earth and impact loads unless it is well anchored and protected (Figure 2.8).

Pipe end sections, sometimes called flared or terminal end sections, are prefabricated metal or precast concrete sections placed onto the ends of small culverts. These sections are used to retain the embankment and improve the aesthetics, but usually do not improve the structural stability of the culvert end.

Headwalls and wingwalls are generally cast-in-place concrete structures commonly constructed on the ends of culvert barrels for the following reasons:

1. To retain the fill material and reduce erosion of embankment slopes,
2. To improve hydraulic efficiency,

3. To provide structural stability to the culvert ends and serve as a counter weight to meet bouyant or uplift forces, and
4. To inhibit piping, which is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a holllow similar to a pipe, thus resulting in possible washing out of fine soil particles along the hollow and failure of the culvert due to erosion.

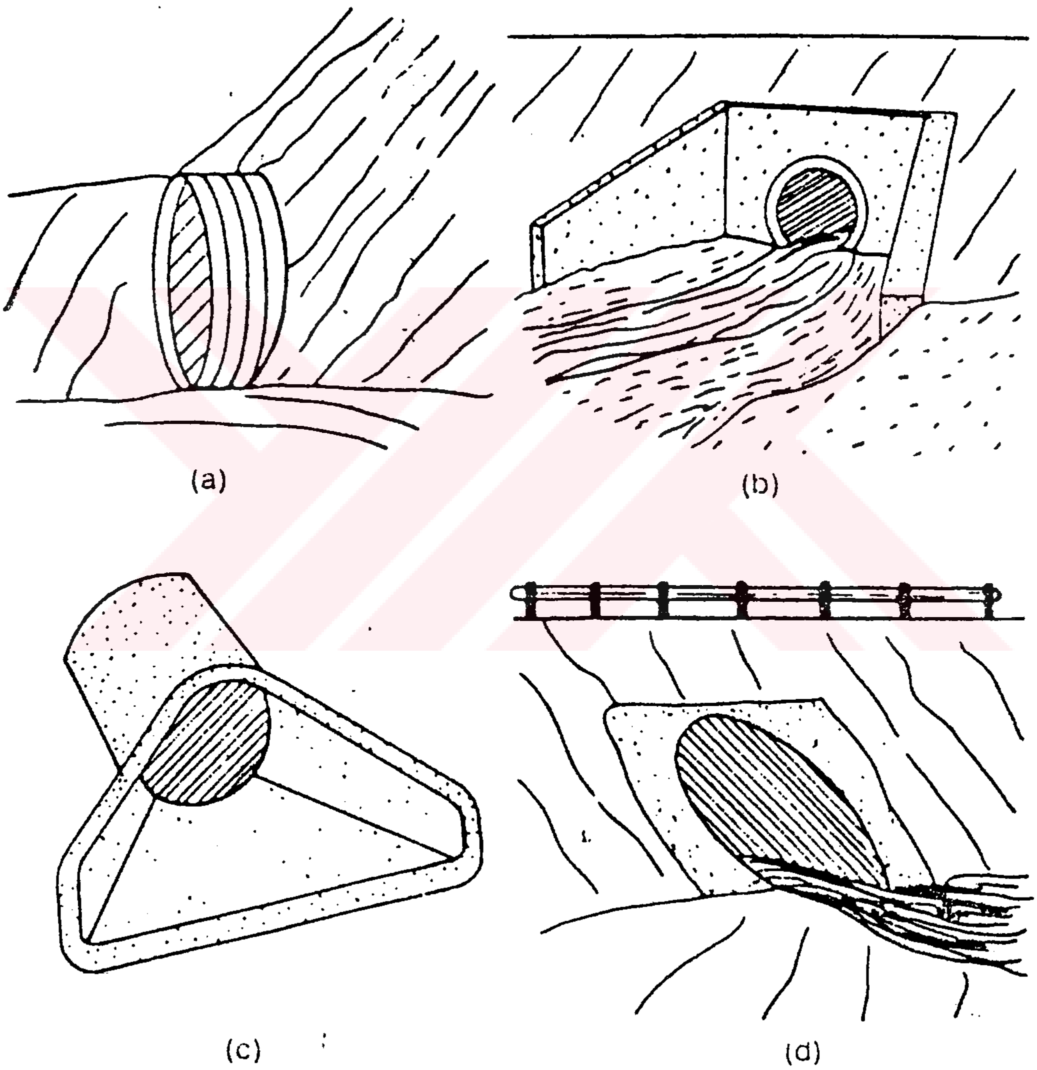


Figure 2.8 Four standard inlet types (schematic): (a) projecting barrel, (b) cast-in-place concrete headwall and wingwalls, (c) precast end section, and (d) end mitered to the slope (ASCE, 1985)

Rounded Lip is another type of treatment which costs little, smooths flow contraction, increases culvert capacity, and reduces the level of ponding at the entrance. The box culvert and pipe headwall standard plans include a rounded lip. The rounded lip is omitted for culverts less than 1200 mm in diameter; however, the beveled groove end of concrete pipe at the entrance produces an effect similar to that of a rounded lip (Highway Design Manual, 1997, p. 820-7).

Moreover, at a location where the culvert would be subject to plugging, a vertical pipe riser generally known as entrance risers in terminology should be considered.

2.5 Basic Principles of Culvert Construction

In order for an efficient and economic operation of a culvert within a drainage system, culverts should be located properly. Culvert location means alignment and grade with respect to both roadway and stream. Proper location is important because it influences adequacy of the opening, maintenance of the culvert, and possible washout of the roadway. There are some basic principles to be obeyed while locating culverts:

The first principle of culvert location is to provide the stream with a direct entrance and a direct exit. Any abrubt change in direction will retard the flow and make a larger structure necessary. A direct inlet and outlet, if not already existing, can be obtained by means of a channel change or a skewed alignment or both (Figure 2.9).

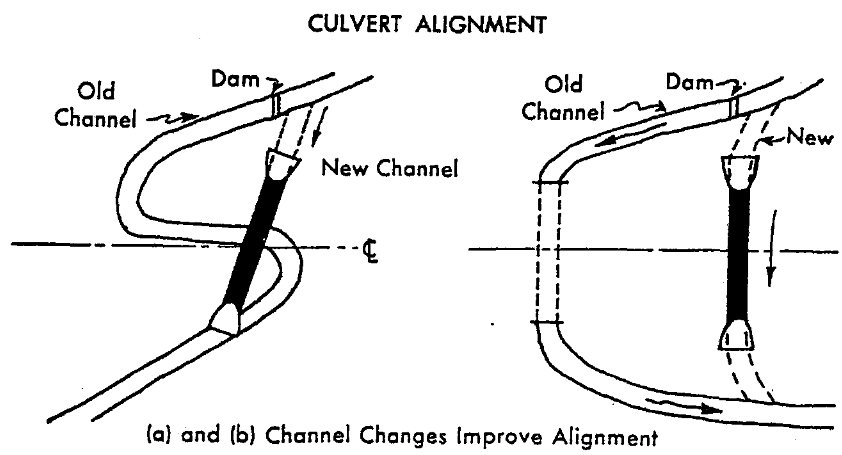


Figure 2.9 Channel changes to provide more direct flow (ARMCO, 1958)

The second principle of culvert location is to use reasonable precaution to prevent the stream from changing its course near the ends of the culvert. If the stream has a tendency to meander, every effort should be made to get it across the roadway as quickly as possible. Otherwise, the culvert may become inadequate, cause excessive ponding and thus it may lead to excessive maintenance of the roadway. Steel end sections, riprap or paving will help protect the banks from eroding and changing the channel (Figure 2.10).

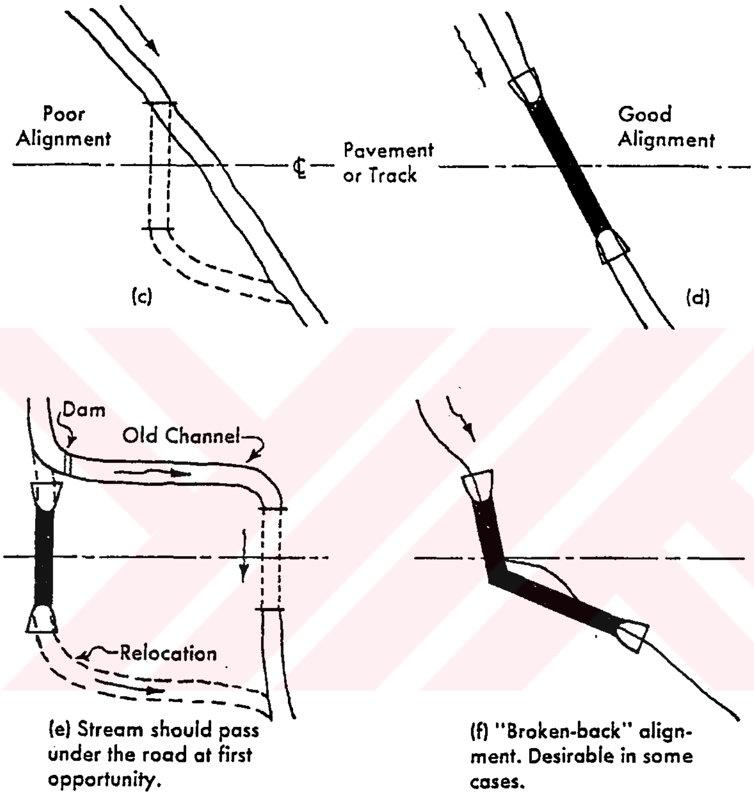


Figure 2.10 Various methods of obtaining correct culvert alignment (ARMCO, 1958)

Moreover, the ideal grade line for a culvert is the one that produces neither silting nor excessive velocities and scour, the one that gives the shortest length, and the one that makes replacement simplest. The slope of a culvert should normally conform as closely as possible to the natural grade of the stream. If the grade is reduced through the culvert, the velocity may be reduced, sediment carried in the water will be deposited at the mouth or in the length of the culvert, and thus, the capacity of the structure will reduce. On the other hand, if the culvert grade is selected greater than that of the natural channel, higher velocities may be resulted through the culvert and at the outlet, and the increased water velocity may cause scouring of the sides and base of the channel at the outlet of the culvert. Changes in grade within the length of the culvert should also be avoided.However, in cases where grade change within the culvert becomes inevitable, the upstream grade should not be greater than the downstream grade in order to prevent the accumulation of sediment within the culvert due to the velocity decrease in water passing through the culvert (Figure 2.11).

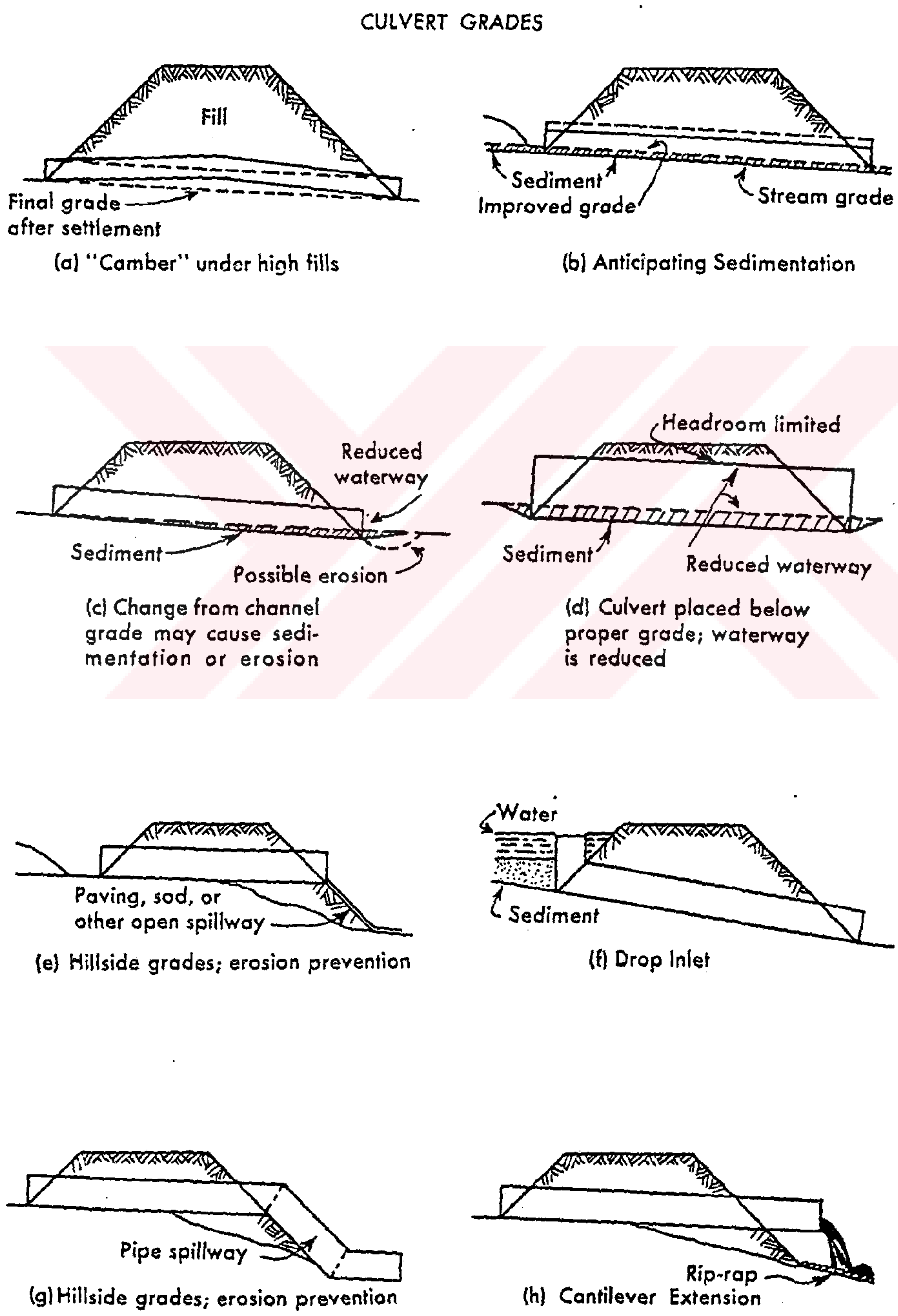


Figure 2.11 Proper culvert grades essential to safe functioning of the structure (ARMCO, 1958)

Furthermore, bottom of the opening in the downstream end of the culvert should be at the same level with the bottom of the stream, otherwise scouring will occur if the opening is higher or the capacity of the culvert will decrease if the bottom of the opening is lower than the bottom of the natural stream.

Natural watercourse should also be leveled at locations where water enters the culvert and gets out of the culvert so that the centerline of the waterbed would seem to continue as a part of the centerline of the culvert.

2.6 Culvert Length

The required length of a culvert depends on the width of the roadway, the height of fill, the slope of the embankment, the slope and skew of the culvert, and the type of end section. Mainly, a culvert should be long enough to prevent its ends from clogging with sediment or with spreading embankment.

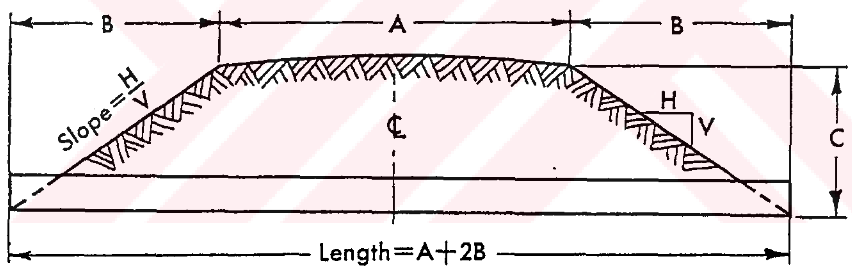


Figure 2.12 Computation of culvert length when flow line is on a flat grade (ARMCO, 1958)

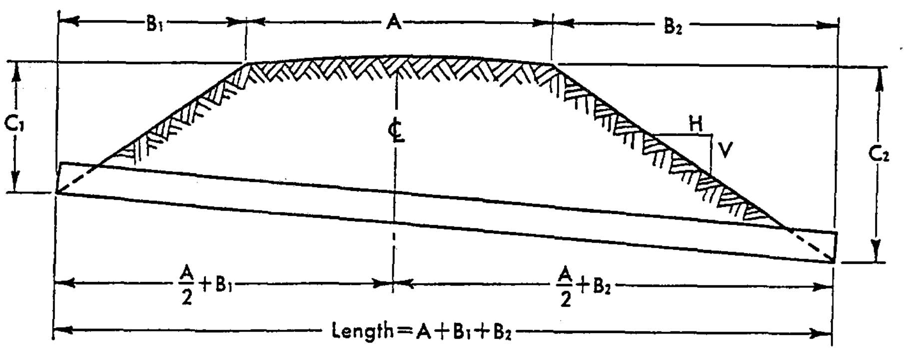


Figure 2.13 Determining culvert length on a steep grade (ARMCO, 1958)

In preliminary design of a culvert, just to have an idea, the length of a simple culvert under an embankment can be determined as folllows:

At the side slopes of the road embankment, the horizontal length of the sloping section is calculated by using the slope ratio and the height of fill, to flow line, at the center of the road; and approximate length of the culvert is computed by adding twice this amount (for two opposite sloping sections) to the width of the road way (and shoulders) (Figure 2.12). If the culvert is on a slope, it may be advisable to compute the sloped length in the manner shown in Figure 2.13. In this case, as the culvert grade will generally be very small compared to the side slopes, the sloped length of the culvert may be taken equal to the horizontal length computed in the figure.

2.7 Bottom Location

The bottom location of a culvert is the location where the invert of the culvert follows the line and grade of the natural stream channel as closely as possible (Figure 2.14). In general, it may be said that this type of installation is one of the best from the hydraulic point of view, even though it may result in the pipe being laid on a curve or a broken grade line. However, there are several disadvantages to this type of location. If the natural gradient of the stream channel is very steep, the exit velocity of the water from the culvert may be high, which may result in problems of erosion and scour. If the bottom of the stream channel lies in a valley, the fill over the culvert may be very high, resulting in extremely long culverts and very heavy loads.

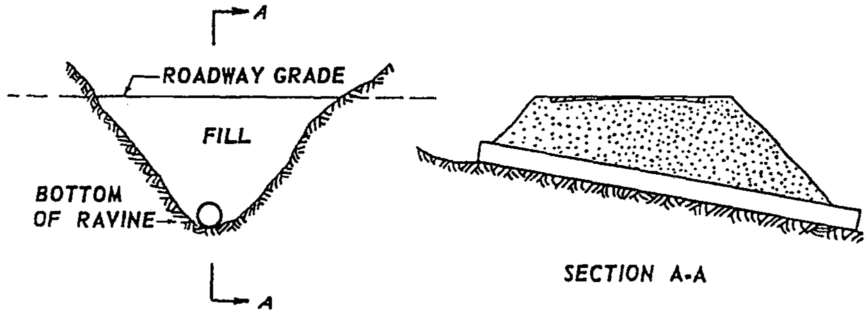


Figure 2.14 Bottom location (Hendrickson, 1958)

A modification of this type of location is sometimes used to permit a reduction in the length and the slope of the culvert pipe. The entrance to the culvert is placed in the bottom of the rectangular channel, but the culvert barrel is in the fill material itself and the outlet will be in the downstream face of the embankment. (Figure 2.15) Depending on the type of material in the fill, it may be necessary to protect the downstream slope against erosion.

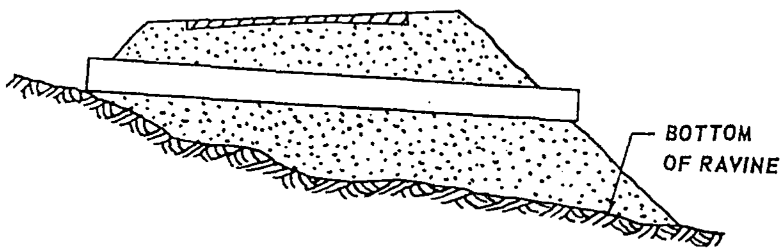


Figure 2.15 Modified bottom location (Hendrickson, 1958)

2.8 Top Location

Sometimes it may be desirable to place the culvert near the top of the embankment fill just beneath the roadway grade. Such an installation is generally described as a top location (Figure 2.16). It permits the use of a shorter length of culvert and the load on the culvert is also considerably reduced.

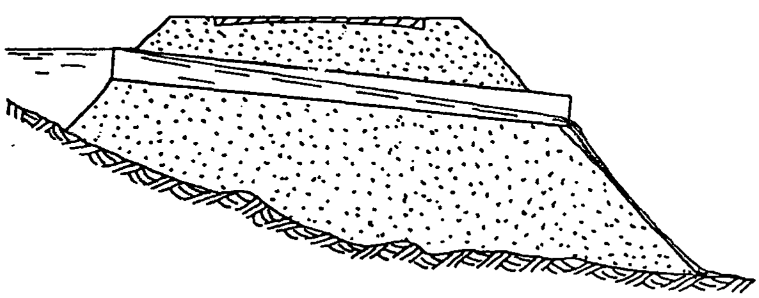


Figure 2.16 Top location (Hendrickson, 1958)

In this type of installation it will be necesssary for the embankment to serve as a dam, raising the level of the upstream pool until water flows through the culvert. Therefore, the embankment must be constructed of material that will insure stability during flood conditions. However, as the outlet of the culvert will not deliver water directly to the natural stream channel, spillways or other types of artificial channels may be needed to carry water from the outlet without eroding the embankment.

2.9 Sidehill Location

Under some conditions, it may be preferable to install the culvert in the disturbed earth of the side hill (Figure 2.17). The load on the culvert due to the weight of the fill can be greatly reduced by placing the culvert in a trench excavated in the side of the hill.

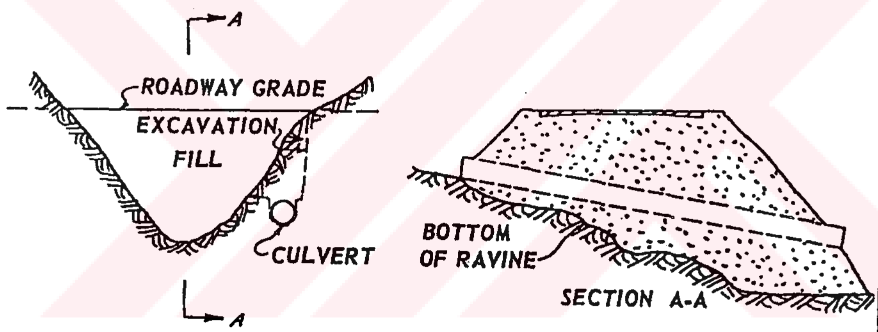


Figure 2.17 Sidehill location (Hendrickson, 1958)

In the sidehill location it is very important to see that the culvert be located entirely in the undisturbed earth of the sidehill. If this is not done, the pipe may be subjected to very heavy loads resulting from the settlement of the newly placed fill and lack of settlement of the sidehill.

CHAPTER THREE
HYDROLOGICAL STUDY

3.1 Surface Hydrology

Regardless of the size or cost of the drainage feature, the most important step prior to hydraulic design is estimating the discharge (rate of flow) or volume of runoff that the drainage facility will be required to convey or control. Cross drainage design, normally requires more extensive hydrologic analysis than is necessary for roadway drainage design. The result of the hydrologic analysis is the projected runoff for the hydraulic design of surface drainage facilities. Runoff is defined as the portion of precipitation that makes its way towards streams, channels, lakes, or oceans as surface or subsurface flow. The design of drainage channels, bridges, culverts, and other engineering structures depends on a knowledge of the amount of runoff that will occur on a given area.

The process of surface runoff begins when precipitation exceeds the requirements of:

- Evaporation
- Vegetal interception
- Infiltration into the soil
- Filling surface depressions (puddles, swamps and ponds). As rain continues to fall, surface waters flow down slope toward an established channel or stream.

Most rainfall-runoff methods require selection of a design storm having a specified recurrence interval. These methods assume that the probability or recurrence interval of the resulting surface water runoff discharge is equal to the probability or recurrence interval of the design storm. Basic assumptions which must be made in order to obtain even a reasonably estimate of design discharge are the following: the rainfall intensity; coefficient of runoff or rate of infiltration; and the time of concentration for critical points in the drainage system.

3.2 Estimating Design Discharge

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The most important step is to establish the appropriate design storm or flood frequency for the specific site and prevailing conditions.

Peak discharge is the maximum rate of flow of water passing a given point during or after a rainfall event. Peak discharge, often called peak flow, occurs at the "peak" of the stream's flood hydrograph (Figure 3.1).

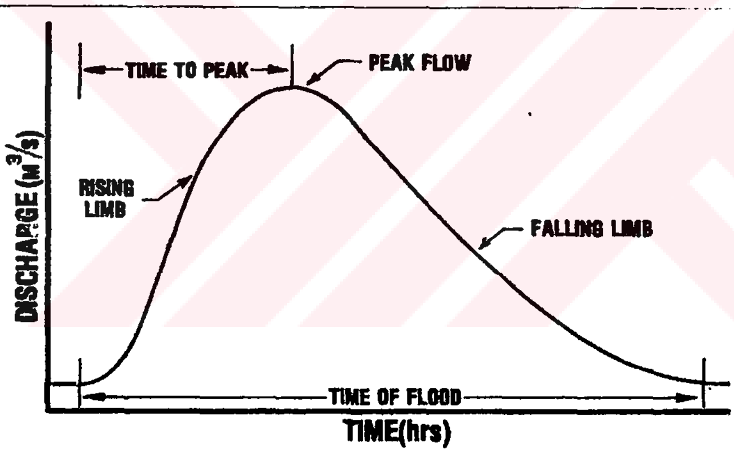


Figure 3.1 Typical Flood hydrograph

Design discharge, expressed as the quantity (Q) of flow in cubic meters per second (m^3/s), is the peak discharge that a highway drainage structure is sized to handle. Peak discharge is different for every storm and it is the highway engineer's responsibility to size drainage facilities and structures for the magnitude of the design storm and flood severity. The magnitude of peak discharge varies with the severity of flood events which is based on probability of exceedance.

Flood severity is usually stated in terms of (Highway Design Manual, 1995, p. 810-2):

- Probability of exceedance, or
- Frequency of recurrence.

Probability of exceedance, the statistical chance of a flood of given magnitude being exceeded in any year, is generally expressed as a percentage. Frequency of recurrence is expressed in years, on the average, that a flood of given magnitude would be predicted.

3.2.1 Selection of Design Flood

There are two recognized alternatives to selecting the design flood frequency (probability of exceedance) in the hydraulic design of bridges and culverts. They are:

- By policy - using a preselected recurrence interval.
- By analysis - using the recurrence interval that is most cost effective and best satisfies the specific site conditions and associated risks.

There are two primary design frequencies that should be considered for culverts:

- A 10% probability flood (10-year) without causing the headwater elevation to rise above the inlet top of the culvert and,
- A 1% probability flood (100-year) without headwaters rising above an elevation that would cause objectionable backwater depths or outlet velocities.

The designer must use discretion in applying the above criteria. Design floods selected on this basis may not be the most appropriate for specific project site locations or conditions. The cost of providing facilities to pass peak discharges suggested by these criteria need to be balanced against potential damage to the highway and adjacent properties upstream and downstream of the site. The selection of a design flood with a lesser or greater peak discharge may be warranted and justified by economic analysis. A more frequent design flood than a 4% probability of exceedance (25-year) should not be used for the hydraulic design of culverts under freeways and other highways of major importance. Alternatively, where predictive data is limited, or where the risks associated with drainage facility failure are high, the greatest flood of record or other suitably large event should be evaluated by the designer.

A number of methods to estimate the amount of runoff are available. Some of these methods aim only to predict the peak flow rate whereas the others produce the entire overland flow hydrograph. They range from the widely used rational formula to the more recently developed computer models that are being upgraded continuously. The method selected should be based on the size of the drainage area, the data available, and the degree of sophistication warranted for the design.

3.2.2 Rational Method

One of the most common methods of estimating runoff from a drainage area is the rational method. The rational method provides a means of converting rainfall intensity into a rate of storm runoff for small drainage areas by application of the following formula:

$$Q = (0.278).C.I.A \tag{1}$$

where

Q = rate of runoff in cubic meters per second,

C = runoff coefficient representing ratio of runoff to rainfall,

I = intensity of rainfall in mm per hour, which may occur during time of concentration,

A = area to be drained, in square kilometers.

The idea behind the rational method is that if a rainfall of certain intensity begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration, when all of the basin is contributing to flow at the outlet. One of the principal assumptions of the rational method is that the predicted peak discharge has the same return period as the rainfall. Another assumption is the constancy of the runoff coefficient during the progress of individual storms and also from storm to storm.

The following steps are utilized in application of the Rational formula for determination of peak flow (Usul, 1994):

1. Estimate the time of concentration of the drainage area (t_c),
2. Estimate the runoff coefficient (C),
3. Select a return period (T_r), and find the intensity of rain that will be equaled or exceeded, on the average, once every T_r years. To produce equilibrium flows, this design storm must have a duration equal to t_c. The desired intensity can be found from Intensity – Duration – Frequency curves of that region using a rainfall duration equal to the time of concentration.
4. Determine the desired peak flow (Q_p).

The runoff coefficient, C, represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage. The runoff coefficient is the least precise variable of the rational method. Its use in the formula implies a fixed ratio of peak runoff rate to rainfall rate for the drainage basin, which in reality is not the case. The proportion of the total rainfall that will reach the storm drains depends on the percent imperviousness, slope, and ponding character of the surface. Values of C vary from 0.05 for flat sandy areas to 0.95 for impervious urban surfaces (Usul, 1994), and considerable knowledge of the catchment is needed in order to estimate an acceptable value. Suggested coefficients for various surface types are given in Table 3.1 and 3.2.

In urban areas, the drainage area usually consists of subareas or subbasins of different surface characteristics. Therefore, a composite analysis is used to take care of the various surface characteristics in the basin. If the areas of subbasins are denoted by A_j and the runoff coefficients of each subbasin are denoted by C_j, the peak runoff is then computed by using the following form of the rational formula;

$$Q_p = (0.278).I.\sum_{j=1}^{n} C_j.A_j \qquad (2)$$

where n is the number of subbasins.

The rainfall intensity, I, is chosen as the maximum intensity for any given frequency of the duration which corresponds to the time of concentration, t_c. The value of rainfall intensity to be selected depends upon the curves for the intensity of rainfall plotted for the local vicinity and the assumed period of occurrence, as well as the period of concentration required for surface runoff to flow from the most distant point in the area under study to the inlet structure or point of collection being considered.

Table 3.1 Runoff coefficients for developed areas (HDM, 1995, c.810)

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.25
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

Table 3.2 Runoff coefficients for undeveloped areas (HDM, 1995, c.810)

Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or wood-land, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface stor-age, high; drainage system not sharply defined; large flood plain storage or large number of ponds or marshes

Given: An undeveloped watershed consisting of;
1) rolling terrain with average slopes of 5%,
2) clay type soils,
3) good grassland area, and
4) normal surface depressions.

Solution:

Relief	0.14
Soil Infiltration	0.08
Vegetal Cover	0.04
Surface Storage	0.06
C=	0.32

Find: The runoff coefficient, C, for the above watershed.

The duration of rainfall required to produce the maximum rate of runoff is known as the time of concentration. An assumption made in some of the synthetic unitgraph methods for estimating peak discharge is that maximum flow results when rainfall of uniform intensity falls over the entire watershed area and the duration of that rainfall is equal to the time of concentration.

After precipitation begins, only part of the watershed contributes to surface water flow at any time t because of the travel time to the watershed outlet. The growth of the contributing area may be visualized as in Figure 3.2,a. If rainfall of constant intensity begins and continues indefinitely, then the area bounded by the solid line labeled t_1 will contribute to streamflow at the watershed outlet after time t_1. Similarly, the area bounded by the line labeled t_2 will contribute to streamflow after time t_2. The boundaries of these contributing areas are called isochrones which may be defined as paths on which all the points have equal travel time, τ, to the outlet. The change in the contributing area as τ increases can be seen from the graph in Figure 3.2,b.

Therefore, the time of concentration can also be defined as the time of flow from the farthest point on the watershed (point D in the figure) to the outlet (point O) so that all of the watershed begins to contribute the discharge at the outlet.

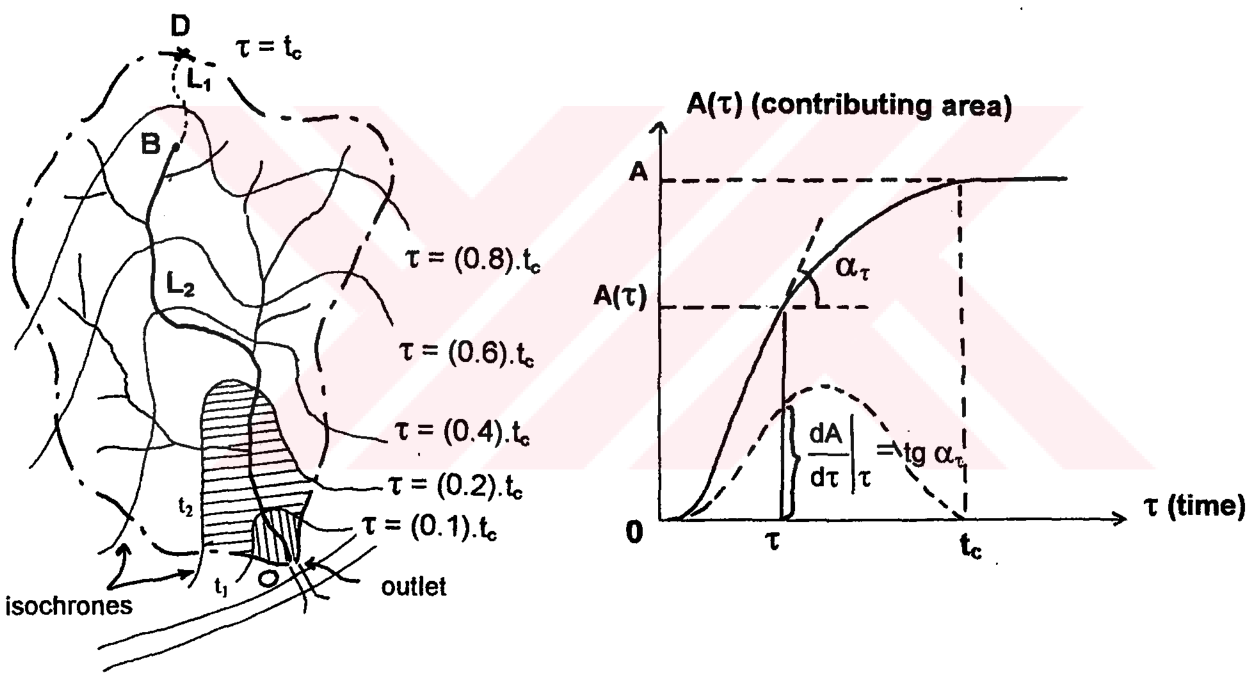


Figure 3.2 (a) Isochrones and concept of time of concentration
b) time-area and time-area-concentration curves (Dooge,1973)

Time of concentration(t_c) is the cumulative sum of three flow times, including:

- Overland
- Channel (ditch or stream)
- Culvert or Storm Drain

The time of overland flow (t_1) is the time required for a particle of water to flow from the most remote point in any section of the drainage area being considered to the point where it enters the drainage system. It can be directly taken from Figure A.1 in Appendix A. The required distance to calculate the time of overland flow can be obtained from 1/25.000 scaled maps. Also, the time of overland flow can be computed from the following formula:

$$t_1(\text{min}) = \frac{L_1}{V_1 x 60} \tag{3}$$

where L_1 is the distance from the most remote point in meters and V_1 is the velocity of overland flow in meters per second obtained from Figure A.2 by using the land slope, soil type and vegetation.

The time of channel flow, t_2, is simply read from the nomogram given in Figure A.3 and the total time of concentration, t_c, is found by adding t_1 and t_2 together.

$$t_c = t_1 + t_2 \tag{4}$$

The time of concentration, t_c, in hour can also be calculated empirically from the following formula known as Kirpich Formula (Dooge, 1973; Chow et al.,1988):

$$t_c(\text{min}) = (0.0195).\left(\frac{L^3}{H}\right)^{0.385} \tag{5}$$

in which H is the elevation difference in meters between the maximum elevation in the basin and the elevation of the drainage structure and L is the lateral distance again in meters between these two points. The same result obtained from Kirpich Formula can also be obtained from the nomograph given in Figure A.4 which has been prepared based on the original equation.

Having estimated t_c, the required rainfall intensity with duration t_c and return period T can be read from intensity-duration-frequency curve. Finally, design peak flow can be calculated by using rainfall intensity I, coefficient of runoff C and drainage area A with the help of rational formula.

3.2.2.1 Positive Features of the Rational Method

An advantage of the rational method is its ease of application in urban areas. Necessary subbasin data can readily be obtained and the required rainfall-duration-frequency curves are usually available or may be developed from published national reports.

3.2.2.2 Negative Features of the Rational Method

A principal limitation of the rational method is that only a peak discharge is produced. Therefore, the rational method cannot be used to calculate the volume of stormwater runoff, a quantity that is often required to evaluate alternative stormwater management measures.

Another negative feature of the rational method is that the time of concentration corresponds to the duration of a storm. The time of concentration is simply the critical time period used to determine average rainfall intensity from intensity-duration-frequency curves. Inasmuch as rainfall is a random event, it is possible that a rainfall event could have a duration equal to the time of concentration of a subbasin. However, it is much more likely that the total duration of a storm will be longer than the time of concentration used in the rational method.

3.2.3 Hydrograph Methods

Generally hydrograph methods are used when dealing with larger drainage areas ($A>15\ km^2$) where the rational method cannot be applied. A hydrograph is the graph of discharge passing a particular point on a stream, plotted as a function of time. The shape of hydrograph changes for different conditions of rainfall and soil characteristics. As water movement in soil starts after the field capacity of soil is satisfied, when there is no rainfall, hydrograph will be a slowly decreasing curve. In a flood condition, as the rainfall occurs, the shape of the hydrograph differs from the curves before and after the flood.

Unit hydrograph of a basin is defined as the hydrograph of surface runoff resulting from 1 cm of excess rainfall generated uniformly over the basin area at a uniform rate during a specified period of time and can be used to obtain the peak discharge resulted from the total excess rainfall having a certain duration and depth. Unit hydrograph theory is first proposed by Sherman in 1932, and it is assumed that unit hydrograph is representative for the runoff process of a basin. There are five assumptions for the unit hydrograph theory (Usul, 1994):

1) Excess rainfall is uniformly distributed within a specified period of time.
2) Excess rainfall is uniformly distributed throughout the whole drainage area.
3) Base time of direct runoff, which is the time from the beginning of surface runoff to its end, is constant for a specified duration of rainfall.
4) The ordinates of the direct runoff hydrograph of a specified duration rainfall are directly proportional to the total amount (depth) of direct runoff, i.e. the amount of excess precipitation.
5) Unit hydrograph is unique for a basin (it reflects the unchanging characteristics of the watershed).

The unit hydrograph for a particular basin may be derived from the recorded hydrograph resulting from a storm which covers the basin uniformly and has a uniform intensity. If the basin is very large, it may never be covered by a uniform density. In such a case, the basin should be divided into tributary subbasins and the unit hydrographs for each of these should be determined separately.

The procedure for obtaining a unit hydrograph from an observed hydrograph is as follows (Usul, 1994):

1) First, rainfall records are studied to select storms giving reasonably high excess rainfall depths, i.e. direct runoff depths (e.g. the storms with depths more than 1 cm).
2) Direct runoff is separated from the other components in the selected storm hydrographs.
3) The volume and depth of direct runoff for each hydrograph are determined.

4) Φ-index is determined to find the duration of excess rainfall which gives the duration of hydrograph.
5) The ordinates of each of the direct runoff hydrographs are divided by the corresponding direct runoff depth to obtain unit hydrograph.
6) The same duration unit hydrograph is obtained from all the different duration hydrographs.
7) An average or representative unit hydrograph for the basin is determined by averaging the peak flows, times to peak, and time bases of the unit hydrographs obtained in the previous steps. Average unit hydrograph is then sketched following the shapes of the individual hydrographs.
8) Finally, the area under the curve should be adjusted to unit depth.

3.2.4 Synthetic Unit hydrographs

Unit hydrographs can be derived only if records of rainfall and their corresponding runoff data are available (Linsley et al., 1988). But there are many catchments for which there are no rainfall and runoff records or there are records but not suitable for unit hydrograph derivation. In these circumstances, hydrographs may be synthesized on the basis of past experience in other areas and applied as first approximations to the ungaged catchments. Hydrographs produced by such a procedure are called synthetic unit hyrographs and they can also be used in estimating the peak discharge produced by a given design storm.

There are mainly three methods commonly used in hydrologic practice for determining a synthetic unit hydrograph for a specified drainage basin:

a) Mockus Method
b) Snyder Method
c) SCS Dimensionless Unit Hydrograph Method

3.2.4.1 Mockus Method

This method should be used for drainage areas having a concentration time t_c less than 30 hours. For larger drainage areas, the method can be applied by dividing the total drainage area into subareas and thus plotting the hydrographs by considering the corresponding delay times .

In Mockus method, the computation is very practical and there is an ease of plotting only triangular hydrographs (SCS, 1972; Chow et al., 1988; Kızılkaya, 1988) (Figure 3.3).

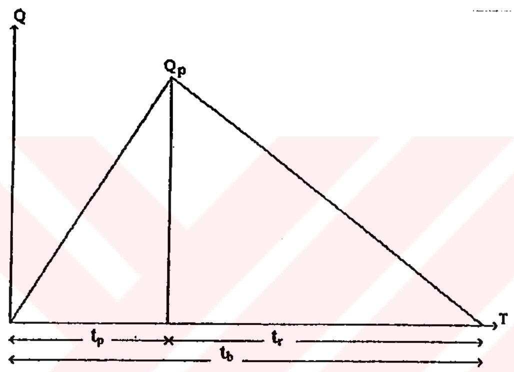


Figure 3.3 Triangular hydrograph

In this method, special importance should be attached to the selection of unit rainfall duration ΔD and the basic citeria for this selection is that $\Delta D \leq (t_c/5)$ where t_c is the time of concentration.

ΔD is generally equal to 1 hour for the first 6 hours of rainfall duration. If t_c is smaller than 3 hours, then ΔD should be taken as 1/2 hour; if it is between 10 and 15 hours, ΔD should be taken as 2 hours and for t_c between 15 and 30 hours ΔD should be taken as 3 hours.

Application procedure for the Mockus method is as follows:

1)Drainage area (A), the longest stream length (L) and harmonic slope (S_h) are determined.

Length of the main course of the channel is generally divided into 10 equal segments and the slope of the each segment, S_i, is calculated. Then, the Harmonic slope is computed by:

$$S_h = \left(\frac{10}{\sum 1/\sqrt{S_i}} \right)^2 \qquad (6)$$

2)Time of concentration, t_c, is calculated from the following formula:

$$t_c(hours) = (0.00032) * \frac{L^{0.77}}{S_h^{\ 0.385}} \qquad (7)$$

where L is the length of the stream in meters.

3)Effective rainfall duration creating the flood is;

$$D(hours) = 2\sqrt{t_c} \qquad (8)$$

and the unit rainfall duration is;

$$\Delta D = t_c / 5 \qquad (9)$$

4)The time from the start of hydrograph to the peak flow (time to peak, t_p) and the time from the peak flow to the end of the hydrograph (t_r) are calculated as follows:

$$t_p(hours) = (0.5) * \ D + (0.6) * t_c \qquad (10)$$

and

$$t_r(hours) = H * t_p \ \text{(Average value of H is 1.67).} \qquad (11)$$

5)Base time, t_b, of the hydrograph, which is the time from the beginning of surface runoff to its end, is computed:

$$t_b = t_p + t_r \cong t_p + 1.67.t_p \cong 2.67 * t_p \qquad (12)$$

6)Peak flow is, then, calculated as follows:

$$Q_p = \frac{K * A * ha}{t_p} (\text{m}^3/\text{sec}) \tag{13}$$

where A is the drainage area in km^2, ha is the depth of direct runoff in mm, t_p is the time to peak in hr and K is a dimensionless coefficient.

7)With the help of Q_p, t_p and t_r, unit hydrograph is plotted. The most important point here is the determination of K and H coefficients. For this determination, the following procedure is applied in basins where rainfall and runoff records are available:

a)At least five hydrographs resulted from uniform rainfall in the drainage area are selected. In each of them, base flow (or groundwater flow) is separated from peak flow, so that direct runoff peak (or surface runoff hydrograph peak) is determined.

b) K coefficient is computed from the following equation:

$$K = \frac{Q_p * t_p}{ha * A} \tag{14}$$

and H coefficient is obtained from the equation below:

$$H = \frac{2 * 0.278 - K}{K} \tag{15}$$

Average values of K and H are 0.208 and 1.67 respectively (Günerman, 1975).

3.2.4.2 Snyder Method

The best known approach to the production of synthetic unit hydrographs was given by Snyder, who selected the three parameters, which are hydrograph base width, peak discharge and basin lag, as the parameters to define the unit hydrograph. In this approach, the catchment characteristics such as area, shape, topography, channel slope, stream density, and channel storage are considered as the parameters affecting the shape of the unit hydrograph. Having followed certain studies on some basins, Snyder defined a standard unit hydrograph for which the rainfall duration t_r (hr) is related to the basin lag t_p (hr) as (Snyder,1938; US Army Corps of Eng.,1959):

$$t_r = \frac{t_p}{5.5} \tag{16}$$

For this standard unit hydrograph, basin lag is given as follows:

$$t_p = (0.75).C_t.(LL_c)^{0.3} \tag{17}$$

where L (km) is the length of main stream, L_c (km) is the distance from basin outlet to a point on the main channel nearest to the centroid of basin area, and C_t is a coefficient for the basin, derived from gaged basins in hydrologically similar regions.

Peak discharge per unit area, q_p ($m^3/sec/km^2$), of the standard unit hydrograph is,

$$q_p = \frac{(2.75)C_p}{t_p} \tag{18}$$

where C_p is another coefficient for the basin, derived from gaged basins in hydrologically similar regions.

C_t and C_p coefficients can be computed from a gaged basin. For this, L and L_c are measured from the topographic map of the basin. From a unit hydrograph of this basin, t_R (hr) duration of effective rainfall, t_{pR} (hr) basin lag, and q_{pR} ($m^3/sec/km^2$) peak discharge per unit area are obtained. If $t_{pR} = 5.5\ t_R$, then t_R, q_{pR} and t_{pR} are accepted as t_r, q_p and t_p respectively, and C_t and C_p are computed using Equations (17) and (18). However, if t_{pR} is quite different from 5.5 t_R, then standard basin lag is determined as (Chow et al., 1988; Usul, 1994):

$$t_p = t_{pR} + (0.25)(t_r - t_R) \tag{19}$$

Then, t_r and t_p values are found by solving Equation (16) and Equation (19) simultaneously. The values of C_t and C_p are then computed from Equations (17) and (18), with $q_{pR} = q_p$ and $t_{pR}=t_p$.

The C_t and C_p values determined in this way can then be used to derive a synthetic unit hydrograph for an ungaged basin in hydrologically similar regions.

The relationship between q_p and q_{pR} of the required unit hydrograph is given as:

$$q_{pR} = \frac{q_p t_p}{t_{pR}} \tag{20}$$

The base time t_b (hr) of the unit hydrograph can be found from the following equation, by assuming a triangular shape and 1 cm depth requirement (Figure 3.4):

$$t_b = \frac{5.56}{q_{pR}} \tag{21}$$

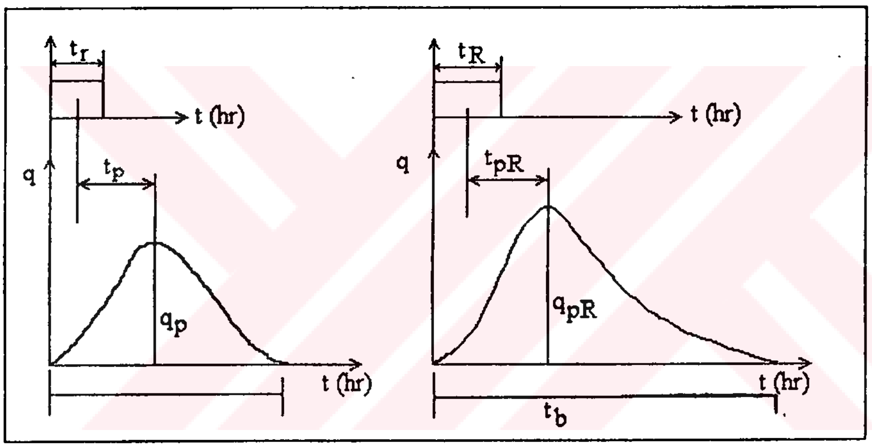


Figure 3.4 Snyder's synthetic unit hydrograph (Usul, 1994)

However, the researches made so far are not enough for the application of Snyder's synthetic unit hydrograph method in Turkey, since the studies do not cover all the regions throughout Turkey. Therefore, one should be very careful in determining synthetic unit hydrograph of a region using the coefficients determined for some other regions, especially if they are not hydrologically similar regions (Usul, 1994) . According to the studies performed in Turkey for only a few regions, similar to the ones performed in U.S.A., the ranges found for these constants are:

For regions with steep slope:	$C_t = 1.35$,	$C_p = 0.69$
For regions with mild slope:	$C_t = 1.5$,	$C_p = 0.63$
For regions with low slope:	$C_t = 1.65$,	$C_p = 0.56$.

3.2.4.3 SCS Dimensionless Unit Hydrograph Method

Input data for the Soil Conservation Service (SCS) dimensionless unit hydrograph method (Soil Conservation Service, 1972) consists of a single parameter, namely the basin lag t_L, which is equal to the lag in hours between the center of mass of excess rainfall and the peak of the unit hydrograph. Time to peak and Peak flow are computed as (SCS, 1972):

$$t_p = (0.5) * t_r + t_L \tag{22}$$

$$Q_p = 484 * A / t_p \tag{23}$$

where t_p is the time to peak of the unit hydrograph in hours, t_r is the duration of excess rainfall in hours or the computation interval, Q_p is the peak flow of unit hydrograph in cfs/in, and A is the area of the subbasin in square miles. The unit hydrograph is interpolated for the specified computation interval and computed peak flow from the dimensionless unit hydrograph shown in Figure 3.5 .

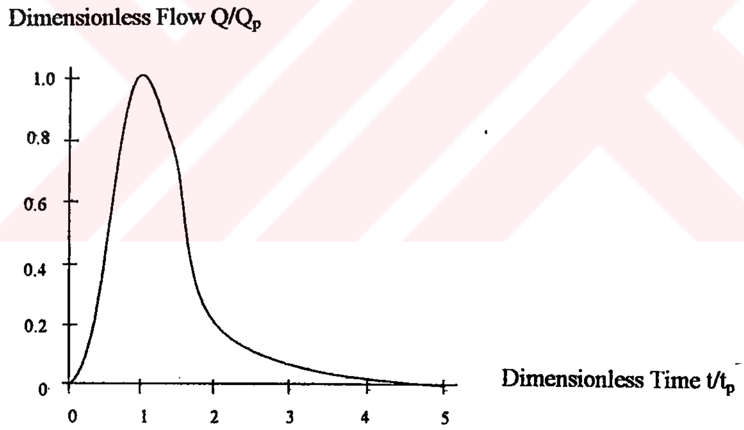


Figure 3.5 SCS dimensionless unit hydrograph (Onuşluel, 1999)

The selection of the computation interval, which is also the duration of the unit hydrograph, is based on the relationship $t_r = 0.2 * t_p$ (SCS, 1972). There is some latitude allowed in this relationship; however, the duration of the unit graph should not exceed $t_r \le 0.25*t_p$. These relations are based on an empirical relationships, $t_L = 0.6*t_c$, and $1.7*t_p = t_r + t_c$ where t_c is the time of concentration of the watershed. The duration should not be greater than $t_r \le 0.29*t_L$.

CHAPTER FOUR
HYDRAULIC STUDY

4.1 Hydraulics of Culverts

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. If a culvert were the same size and shape as the channel on which it is placed, then its hydraulic design would be relatively easy. However, a culvert normally has the effect of a constriction to the stream flow. The efficient design of this constriction may be very difficult depending on certain governing factors. These factors and some of their effects are as follows:

4.1.1 Design Flood Discharge

As mentioned earlier, the flood discharge used in culvert design is usually estimated on the basis of a preselected recurrence interval, and the culvert is designed to operate in a manner that is within acceptable limits of risk at that flow rate.

4.1.2 Headwater Elevation

Any culvert which constricts the natural stream flow will cause a rise in the upstream water surface to some extent. The total flow depth in the stream measured from the culvert inlet invert is termed headwater. Design headwater elevations and selection of design floods should be based on these risk conditions:

1. Damage to adjacent property;
2. Damage to the culvert and the roadway;
3. Traffic interruption;
4. Hazard to human life; and
5. Damage to stream and floodplain environment.

Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all culverts. In urban areas, the potential for damage to adjacent property is greater because of the number and value of properties that can be affected. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly traveled routes.

However, it is not always economical or practical to utilize all the available head. This applies particularly to situations where debris must pass through the culvert, where a headwater pool cannot be tolerated, or where the natural gradient is steep and high outlet velocities are objectionable. The available head may be limited by the fill height, damage to the highway facility, or the effects of ponding on upstream property. Full use of available head may develop some vortex related problems and also develop objectionable velocities resulting in abrasion of the culvert itself or in downstream erosion. In most cases, provided the culvert is not flowing under pressure, an increase in the culvert size does not appreciably change the outlet velocities.

4.1.3 The Depth of the Headwater Pond

For a given design flood, the depth of the pond formed at the entrance is a function of the size and shape of the culvert. The manner in which flow takes place in the culvert is affected by the head of water available at the inlet. The entrance to a culvert can be considered as hydraulically submerged when the depth above the invert of the inlet is greater than about 1.2 times its diameter.

4.1.4 Tailwater

Tailwater is the flowdepth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full. As its depth or height is dependent upon the downstream topography and other influences, a field of inspection of the downstream channel should be made to determine whether there are obstructions which will influence the flow depth.

4.1.5 The Depth of the Tailwater Pond

In order to prevent possible damages caused by the tailwater at the outlet of the culvert, a pond with a certain depth is formed. However, there is one thing to be considered here such that the culvert will usually run full when both the inlet and outlet are submerged during the design flood, although there are certain instances when it will not behave so. On the other hand, if the outlet is not submerged, it is still possible for the culvert to run full if the inlet is submerged and the culvert is long.

4.1.6 Outlet Velocity

The outlet velocity of highwater culverts is the velocity measured at the downstream end of the culvert and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed to scour and bank erosion for a limited distance downstream from the culvert outlet.

Variation in shape and size of a culvert seldom has a significant effect on the outlet velocity except at full flow. The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. If the outlet velocity of a culvert is believed to be detrimental and it cannot be reduced satisfactorily by changing the barrel roughness or adjusting the barrel slope, it may be necessary to use some type of outlet protection or energy dissipation device. Inspection of existing culverts in the area will be helpful in making this judgement.

4.1.7 The Type of Entrance

If the culvert has a poorly designed entrance, then considerable turbulence will occur at the inlet. Culverts with flared or wing-type entrances are much more efficient then ones with straight square-edge headwalls or where the culverts project into the headwater pond. For instance, culverts with submerged inlets having flared entrances may normally flow full whereas those with poor inlets will not. Good entrance design is a most impotant feature in the design of short culverts and long culverts on steep slopes, but it is of less importance in the design of long culverts on gentle slopes.

4.1.8 The Roughness of the Interior Walls

A culvert with rough interior walls, which runs full, will discharge less water than a smooth one also running full, since much of the energy head is used up in overcoming the resistance to flow. The longer the culvert, the more important is the roughness factor. This fact can easily be seen from the following friction loss formula in a conduit flowing full:

$$h_f = \lambda.\frac{L}{D}.\frac{V^2}{2g} \qquad (24)$$

where h_f is the friction loss, λ is friction loss coefficient, L is the length of culvert, D is the diameter of circular culvert and V is the average velocity within the culvert flowing full.

4.1.9 The Length of the Culvert

As discussed above, the length of the culvert defines whether or not the type of entrance and the roughness of the interior walls are major or minor features of the hydraulic design.

4.1.10 The Slope of the Culvert

When a culvert with a free outlet is flowing full, the difference in elevation between its inlet and outlet is available as energy head. If the culvert slope is increased, this head, and therefore, the flow in culvert will definitely increase. In case that the culvert is not flowing full, if the culvert slope is increased, the discharge does not increase, since the depth of flow decreases while the velocity increases.

4.2 Hydraulics of Culverts In Inlet and Outlet Control

It can be easily seen that the manner in which the hydraulic flow occurs in a culvert is dependent on a number of factors. Culverts can flow under inlet control or outlet control. Inlet control means that the discharge capacity is controlled at the culvert entrance. Under inlet control, the cross-sectional area of the culvert barrel, the inlet configuration or geometry and the amount of headwater (HW) or ponding are of primary importance. In inlet control, the roughness and length of the culvert barrel and outlet conditions, including depth of tailwater, are not factors governing the capacity.

Inlet control for culverts may occur in two ways:

a) Unsubmerged – the headwater is not sufficient to submerge the top of the culvert and the culvert inlet slope is steep (can sustain supercritical flow). The culvert inlet effectively acts like a weir. (Figure 4.1-a)
b) Submerged – the headwater submerges the top of the culvert but the barrel does not flow full. The culvert inlet acts like an orifice. (Figure 4.1-b)

In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation:

$$Q = C_d . A . \sqrt{2gh} \qquad (25)$$

where Q is the flow, C_d is the orifice coefficient, A is the wetted area, g is the gravitational constant and h is the head on culvert measured from barrel centerline.

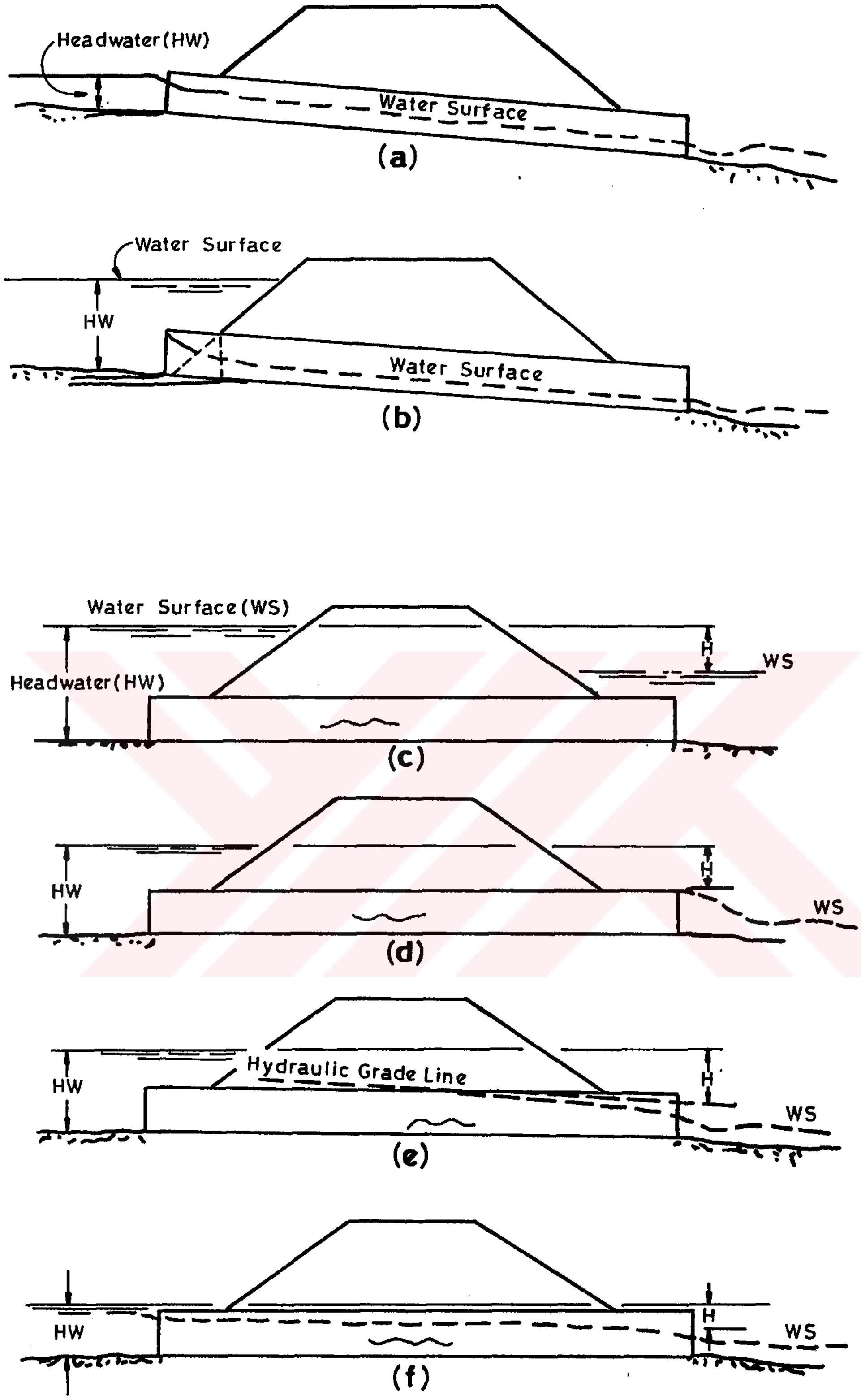


Figure 4.1 Inlet and outlet conditions for culverts: inlet control: (a) projecting inlet end – unsubmerged, (b) projecting or mitered inlet – submerged; and outlet control: (c), (d), (e), and (f)

The orifice coefficient, C_d, varies with head on the culvert as well as the culvert type and entrance geometry. Inlet control nomographs for several culvert materials, shapes, and inlet configurations are given in Appendix A. The nomographs are recommended for use, rather than the above equation, due to the uncertainty in estimating the orifice coefficient, and because they are applicable to a wide range of depths. Equation (25) is applicable only when the ratio of water depth to culvert height (diameter) is greater or equal to 2.

Outlet control will govern if the headwater is deep enough, the culvert slope is sufficiently flat, and the culvert is sufficiently long. There are three types of outlet control flow conditions:

a) The headwater submerges the culvert top, and the culvert outlet is submerged under the tailwater. The culvert will flow full. (Figure 4.1-c)
b) The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater. (Figure 4.1-d,e)
c) The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth. (Figure 4.1-f)

The capacity of the culvert under outlet control is calculated using the principle of the conservation of energy (Bernoulli's Equation). An energy balance is determined between the headwater at the culvert inlet and at the culvert outlet, which includes inlet losses, friction losses, and the velocity head (Figure 4.2). The equation is then expressed as:

$$H = h_e + h_f + h_v \tag{26}$$

where H is the total energy head in meters, which is the head or energy required to pass a given quantity of water through a culvert flowing in outlet control; h_e is the entrance head losses (minor losses) in meters, which depends upon the geometry of

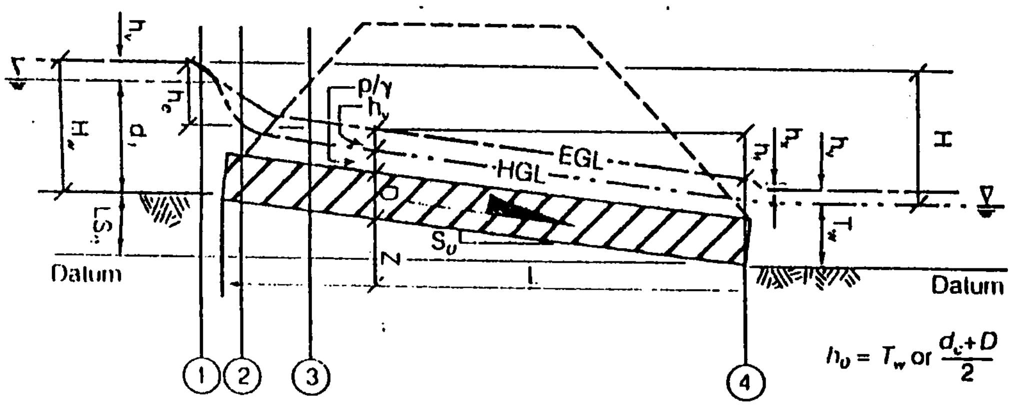


Figure 4.2 Hydraulics of a culvert under outlet condition: L = culvert length, S_0 = culvert slope, H_w = headwater depth, h_v = velocity head, h_e = head loss at entrance, Z = distance from datum line, D = culvert diameter or rise, p/γ = pressure head, HGL = hydraulic grade line, EGL = energy grade line, T_w = tailwater depth, h_x = head loss at exit, and h_f = friction loss in culvert

the inlet edge; h_f is the friction losses in meters and h_v is the velocity head, which is equal to $V^2/2g$, in meters.

For entrance losses the governing equation is:

$$h_e = K_e \cdot \left(\frac{V^2}{2g}\right) \tag{27}$$

where K_e is the entrance loss coefficient. Typical inlet loss coefficients recommended for use are given in Table 4.1 .

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows (in SI units):

$$h_f = \frac{19.5 n^2 L}{R^{1.33}} \left(\frac{V^2}{2g}\right) \tag{28}$$

Table 4.1 Hydraulic Data for Culvert: Culvert Entrance Losses (Walesh, 1989)

Type of Entrance	Entrance Coefficient, K_e
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall & 45° Wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square edged at crown	0.70

where n is the Manning's coefficient (Table 4.2), L is the length of the culvert, and R is the hydraulic radius.

Table 4.2 Average values for Manning's roughness coefficient (n) (HDM, 1995, c.860)

Type of Channel	n value
Unlined Channels:	
Clay Loam	0.023
Sand	0.020
Gravel	0.030
Rock	0.040
Lined Channels:	
Portland Cement Concrete	0.014
Air Blown Mortar (troweled)	0.012
Air Blown Mortar (untroweled)	0.016
Air Blown Mortar (roughened)	0.025
Asphalt Concrete	0.018
Sacked Concrete	0.025
Pavement and Gutters:	
Portland Cement Concrete	0.015
Asphalt Concrete	0.016
Depressed Medians:	
Earth (without growth)	0.040
Earth (with growth)	0.050
Gravel	0.055

Combining the above three equations and simplifying gives (in SI units):

$$H = \left(K_e + 1 + \frac{19.5n^2 L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \qquad (29)$$

The Equation (29) can be used to calculate the culvert capacity directly when the culvert is flowing under outlet conditions a or b as shown on Figure 4.1 . For conditions c or d, the hydraulic grade line at the outlet is approximated by averaging the critical depth and the culvert diameter, which is used if the value is greater than the tailwater depth (TW) to compute headwater depth (HW).

A series of outlet control nomographs for various culvert materials and shapes have also been developed (Appendix A). When determining culvert dimensions, either this outlet control nomographs or the above equation can be used to calculate the headwater requirements.

Furthermore, there are certain flow conditions commonly explained in books related to General Directorate of Highways (KGM) in Turkey. Generally, four flow types are mentioned in such books, even though a different flow condition other than these four flow types may occur throughout a culvert (Çağlarer, 1972).

The first type of these flow conditions is named as Type-I flow and the headwater is not sufficient to submerge the entrance of the culvert (i.e., HW<1.2D), the slope of the culvert is less than the critical slope and the tailwater is equal to or less than the critical depth in this type of flow. This type of flow is the same as the flow condition in Figure 4.1-f explained above, where outlet control governs. This general type is common for flat floodplains where natural stream channel is wide and has a mild slope. The governing equation for this type of flow is the following equation:

$$HW = d_c + V_c^2 / 2g + h_e + h_f - S.L \qquad (30)$$

where d_c is the critical depth, V_c is the critical velocity, h_e is the entrance loss calculated as above, h_f is the friction loss along the culvert, S is the slope and L is the length of the culvert.

The second type of flow is named as Type-II flow and in this type of flow, where outlet control governs, the entrance of the culvert is not submerged by the headwater (i.e., HW<1.2D), the slope is less then the critical value and the tailwater depth, TW, is greater than the critical depth and less than the height of the culvert opening (i.e., $TW>d_c$ and TW<D). The governing equation in such a case is as follows:

$$HW = TW + V_{TW}^2 / 2g + h_e + h_f - S.L \qquad (31)$$

where V_{TW} is the velocity at the culvert outlet.

The third type of culvert flow is Type-III flow which is identical to the flow condition mentioned in Figure 4.1-a above where inlet control will govern. In this type, the headwater is insufficient to submerge the entrance of the culvert (i.e., HW<1.2D) and the slope of the culvert is equal or greater than the critical slope for this culvert (i.e., $S>S_c$). This condition is common in culverts located in a mountainous territory. There are certain nomographs prepared to determine the headwater depth in this type of flow.

The fourth type of flow is named as Type-IV flow and in this type, the headwater submerges the entrance of the culvert (i.e., HW>1.2D) and the tailwater depth is greater than the height of the culvert (i.e., TW>D). This case is the same as the condition in Figure 4.1-c mentioned above and the governing equation is as follows:

$$HW = H + TW - S.L \qquad (32)$$

where $H = h_v + h_e + h_f$. Here, h_v is equal to $(V^2/2g)$ and V is the velocity within the culvert.

In the second alternative of this type of flow, the headwater submerges the entrance of the culvert (i.e., HW>1.2D), but the tailwater depth is less than the height of the culvert (i.e., TW<D). The governing equation in this case is:

$$HW = H + P - S.L \qquad (33)$$

where H is the same as above and P is an approximated depth value. If $TW<d_c$, then P is $1/2*(d_c+D)$, otherwise P is taken as equal to the tailwater depth, TW.

CHAPTER FIVE
CULVERT DESIGN

5.1 Design Considerations
5.1.1 Culvert Sizing

Factors to be considered in the sizing of the culvert include minimum design frequency, minimum culvert size, public safety, maintenance, allowable frequency of inundation of upstream and downstream properties (for minimum damage to adjacent properties) and potential for debris accumulation.

The minimum design frequency for culverts is often specified by city or county governments or by state highway departments. Frequency often is specified in terms of a maximum headwater depth for a given return frequency flood. It is essential for the engineer to recognize that the design frequency must be selected for each culvert separately after performing certain studies in that location. For example, a 10-year culvert may be acceptable under certain conditions, whereas a culvert with much greater than 100-year capacity may be required in others.

For economic considerations, minimum culvert size to carry the design discharge is generally prefered. However, public safety should take the most important place in design considerations. For example, it will be very dangerous to increase risks in the design of a culvert at the upstream or downstream of which there is a nuclear power facility or a hospital. One more consideration in design process is to provide minimum damage to not only human life but also to private or public properties nearby.

5.1.2 Debris Control

Debris potential of the tributary basin is another important consideration when determining the minimum acceptable size for a culvert. Debris, if allowed to accumulate either within a culvert or at its inlet, can adversely affect the hydraulic performance of the facility. Damage to the roadway and to upstream property may result from debris obstructing the flow into the culvert. Coordination with district maintenance forces can help in identifying areas with high debris potential and in setting requirements for debris removal where necessary.

The use of any device that can trap debris must be thoroughly examined prior to its use. In addition to the more common problem of debris accumulation at the culvert entrance, the use of safety end grates or other appurtenances can also lead to debris accumulation within the culvert at the outlet end. Evaluation of this possibility, and appropriate preventive action, must be made if such end treatment is proposed. Localities often stipulate that, for example, the engineer should assume 25% debris blockage when computing the required size of the culvert. Under certain conditions, greater than 25% debris blockage may be justified.

There are two methods of handling debris (Highway Design Manual, 1997, p. 820-3):

(1) Passing Through Culvert. If economically feasible, culverts should be designed to pass debris. Culverts which pass debris often have a higher construction cost. On the other hand, retaining solids upstream from the entrance by means of a debris control structure often involves substantial maintenance cost. An economic comparison which includes evaluation of long term maintenance costs should be made to determine the most reasonable and cost effective method of handling.

(2) Interception. If it is not economical to pass debris, it should be retained upstream from the entrance by means of a debris control structure. If drift and debris are retained upstream, a riser or chimney may be required. This is a vertical extension to the culvert which provides relief when the main entrance

is plugged. If debris control structures are used, access must be provided for maintenance equipment to reach the site.

However, debris problems do not occur at all suspected locations. It is often more economical to construct debris control structures after problems develop. An assessment of potential damage due to debris clogging if protection is not provided should be the basis of design.

5.1.3 Velocity Limitations

When designing culverts, both the minimum and maximum velocities should be considered. A minimum velocity of flow is required to assure self-cleaning and this minimum velocity is normally taken to be between 0.5-1.0 meters per second at the outlet.

The maximum velocity in a culvert is controlled by two factors, the channel protection provided at the outlet and the maximum allowable headwater. If the outlet velocities are less than a recommended value (based on specific channel erosion factors) then only minimal protection is required due to the currents generated by flow transitions. As outlet velocity is increased, however, additional protection is required such as more extensive riprap or an energy dissipator structure (U.S. Department of the Interior, 1976).

5.1.4 Headwater Depth

Maximum headwater depths are normally determined by local governments, generally in terms of a particular factor times the culvert diameter or culvert rise dimension for shapes other than round. In Turkey, the limiting values that correspond to allowable headwater depths are generally determined such that the headwater resulted from a 10-year flood will not exceed 90 percent of the culvert diameter or culvert rise, while the one generated by a 100-year flood will not exceed the minimum of the fill height or three times the culvert rise. Considerations governing headwater selection are similar to those described above for design return frequency.

Essentially, this determination should be based upon the consequences to upstream properties for a variety of floods for different headwater depths. There is a general tendency in various culvert engineering applications to down-size the conveyance capacity of a culvert for a given headwater depth in an attempt to save money; however, such a change would increase flood risks to established upstream properties that have developed 'reliance' (as a legal and technical term) on the existing conveyance capacity. Decreasing culvert capacity under these conditions leaves the culvert engineer and the contractor liable for damages in the event where an occurring flood damages upstream properties due to the use of an insufficient culvert.

5.1.5 Structural Design

All culverts should be designed to withstand minimum loading requirements in accordance with the corresponding specifications.

5.2 Culvert Design Methodology

There are three general methods in use for determining the required size of a culvert:

1) Inspection of an old structure at the site, or structures up and downstream,
2) Use of an empirical formula to determine directly the size of opening required,
3) Use of a formula to determine the amount of water reaching the culvert, then a second procedure to determine the size of culvert required to carry this amount of water.

5.2.1 Determining Size by Inspection

One of the most practical ways of determinig proper size is by inspection of an existing structure at the site, even though its size may have been established by guess. In this method, the size and shape and the condition of the channel above and below are noted and it is investigated whether the existing culvert was adequate at peak flood flow condition over a period of twenty-five years or more. Finally, the dimensions of the culvert to be constructed are determined by considering this obtained information about the existing structures at the site.

5.2.2 Determining Size by Use of An Empirical Formula

Because of its simplicity in giving culvert size directly, the Talbot Formula is commonly used in determining the size. It is an empirical formula based on a large number of observations. It does not take into account the intensity of rainfall, velocity of flow or other rational factors. The maximum rainfall for these observations is not known, but is assumed to have been about 4 inches per hour. The velocity of flow was variable, something less than 10 ft per second (ARMCO, 1958).

The Talbot formula gives the area of the culvert opening directly as follows (ARMCO, 1958):

$$A = C\sqrt[4]{M^3} \tag{34}$$

where A is the waterway necessary, in sq ft; M is the area drained, in acres; and C is a coefficient.

The coefficient C depends upon the contour of the land drained, and the following values are recommended for various conditions of topography (ARMCO, 1958):

C = 1, for steep and rocky ground with abrupt slopes.

C = 2/3, for rough hilly country of moderate slopes.

C = 1/2, for uneven valleys, very wide as compared to length.

C = 1/3, for rolling agricultural country where the length of valley is three or four times the width.

C = 1/5, for level district not affected by accumulated snow or severe floods. For still milder conditions or for subdrained lands, C coefficient is decreased as much as 50 percent; but it is increased for steep side slopes or where the upper part of the valley has a much greater fall than the channel of the culvert.

5.2.3 Size Determination To Carry Design Discharge

The third and the most reliable method for the design of a culvert is to use a formula to determine the size required to carry the predetermined amount of water reaching the culvert entrance. The culvert is generally designed to discharge:

a) a 10-year flood without static head at entrance,and
b) a 100-year flood utilizing the available head at entrance.

The permissible height of water at the inlet controls hydraulic design. This should be taken into consideration for each site in which ponding is allowed (in headwater pond), based on the following risk conditions:

1. Risk of overtopping the embankment, and risk to human life,
2. Potential damage to the roadway, due to saturation of the embankment, and pavement disruption due to freeze-thaw.
3. Traffic interruptions,
4. Damage to adjacent or upstream property, or to the channel or flood plain environment,
5. High outlet velocities, and scour and erosion,
6. Injurious deposition of bed load, and clogging by debris.

The Bureau of Public Roads recommends the following general procedure for culvert design (Ritter & Paquette, 1978):

1. Select the average frequency of the design flood (generally T = 10 and T = 100 years, as mentioned early).
2. Estimate the peak discharge of the design flood.
3. Obtain all site data. Plot a roadway cross-section at the culvert site and a stream channel profile.
4. Establish the culvert invert elevations at inlet and outlet, the culvert length and the invert slope.
5. Determine the allowable headwater depth (or depths for 10-year and 100-year flood discharges separately).

6. Compute the depth of flow in the stream channel, including the flood plain, for the design flood, and determine the tailwater depth.
7. Select one or more appropriate culvert types. Compute an approximate barrel area ($A_b = Q/10$) to guide selection of type and possible number and sizes of multiple barrels. Compute the discharge rate (Q) Per barrel if a multiple culvert is used.
8. Select the capacity chart for the culvert and entrance type to be considered. By using the selected chart, check the culvert capacity considering allowable headwater depth values for design discharges.

5.3 An Alternative Design Procedure

Culverts are routinely designed to meet specified headwater, tailwater and other conditions. A seven-step culvert design procedure, which is based on the use of design criteria and corresponding design charts prepared by the U.S. Bureau of Public Roads and the U.S. Department of Transportation, has been obtained by compiling from national and international literature. This stepwise procedure is as follows:

Step 1. List design data.

(a) Design discharge Q, in cubic meters per second, with average return period (i.e., Q_{10} or Q_{100}, etc.).

(b) Approximate length L of culvert, in meters. For this determination, the method explained in Section2.6 could be used.

(c) Slope of culvert. (If grade is given in percent, convert to slope in meters per meter).

(d) Allowable headwater depth, in meters, which is the vertical distance from the culvert invert at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert. This allowable depth can be determined as explained in Section 5.1.4 .

(e) Mean and maximum flood velocities in natural stream.

(f) Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2. Determine the first trial size culvert.

Since the procedure given is based on trial and error approach, the initial trial size can be determined in several ways:

(a) By arbitrary selection.

(b) By using an approximating equation such as $Q/10 = A$ (as proposed by the Bureau of Public Roads) from which the trial culvert dimensions are determined.

(c) By using inlet control nomographs for the culvert type selected. If this method is used, an HW/D ratio must be assumed, such as HW/D = 1.5, and using the given Q a trial size is determined.

(d) By using an empirical formula to determine directly the area of opening required at the entrance (For example; Talbot Formula as explained in Section 5.2.2).

If any trial size is too large because of limited embankment height or availability of pipe, multiple culvert barrels may be used by dividing the discharge equally among the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height may also be considered. Final selection should be based on an economic analysis.

Step 3. Find headwater depth for trial size culvert.

(a) Assuming INLET CONTROL:

1. Using the trial size from Step 2, find the headwater depth, HW, by using the appropriate inlet control nomograph (prepared either in Metric or British units). Tailwater (TW) conditions are to be neclected in this determination. HW in this case is found by multiplying HW/D obtained from the nomographs by the height of culvert, D.
2. If HW is greater than allowable, try another trial size of culvert until HW becomes acceptable for inlet control, before computing HW for outlet control.

(b) Assuming OUTLET CONTROL:

1. Approximate the depth of tailwater (TW), in meters, above the invert at the outlet for the design flood condition in the outlet channel (This approximate value for tailwater depth can easily be obtained from Stage-Discharge curves prepared for this particular stream channel). The idea behind that is to keep the natural flow conditions that were valid before placing a contraction to the natural channel and to provide the stream with previous natural flowdepths after constructing the culvert.
2. For tailwater, TW, elevation equal to or greater than the top of the culvert at the outlet, set h_0 equal to TW and find HW by the equation given below:

$$HW = H + h_0 - L.S_0 \tag{35}$$

For TW > D, h_0 = TW and H is obtained by using corresponding outlet control nomographs.

3. For tailwater elevation less than the top of the culvert at the outlet, find headwater by the same equation given above, but this time, setting h_0 equal to $(d_c+D)/2$ or TW, whichever is greater (where d_c is the critical depth in meters and D is the height of culvert opening again in meters). Originally, $(d_c+D)/2$ is used for tailwater depths below the critical depth; however, in design process, it will be proper to take the greater of the above values rather than the exact TW depth values between $(d_c+D)/2$ and d_c in order to be on the safe side, since a supercritical flow at the culvert outlet is not desired.

(c) Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

(d) If outlet control governs and the HW is higher than acceptable, select a larger trial size and find HW as instructed under Step 3b (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a).

Step 4. Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 5. Compute outlet velocities for size and types to be considered in selection and determine need for channel protection as explained in Section 5.1.3 .

(a) If outlet control governs in Step 3c above, outlet velocity is equal to Q/A_0, where A_0 is the cross-sectional area of flow in the culvert barrel of the outlet. If d_c or TW is less than the height of the culvert barrel, use A_0 corresponding to d_c or TW depth, whichever gives the greater area of flow (Here, there may be a misunderstanding such that smaller area gives a higher velocity, so it may seem to be essential to take this area into consideration. However, the greater value is used because there is no possibility of having a TW depth less than the critical depth, d_c, after the design, based on the above design considerations. Moreover, d_c is used in the area A_0 computations even though it had been stated that TW would be made equal to $(d_c+D)/2$ if it is less than the critical depth. By doing so, it is again aimed to be on the safe side). A_0 should not exceed the total cross-sectional area A of the culvert barrel.

(b) If inlet control governs in Step 3c, outlet velocity can be assumed to be equal to mean velocity in open channel flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Step 6. Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification.

Step 7. Assess flood hazard posed by recommended culvert to upstream and downstream properties, particularly if a change in the conditions, such as increased flood levels, is going to occur.

CHAPTER SIX
ILLUSTRATIVE EXAMPLES

6.1 An Illustrative Example For A Single-Barrel Box Culvert

❖ *Aydın – Denizli Highway* Aydın – Kuyucak section

Station-Km: 33 + 430

Given:

Drainage Area : 0.521 km^2

Invert upstream : 139.22 m

Invert downstream : 138.00 m

Height of Fill : 4.5 m

Length of Culvert : 61 m (~200 ft)

Terrain Type : A rough hilly country of moderate slopes

End Treatment : 45° wingwall with grooved edge

Q_{10} = 5.32 m^3/sn = 187.8 ft^3/sn Tailwater depth = 1.6 m (for Q_{10})

Q_{100} = 7.68 m^3/sn = 271.2 ft^3/sn Tailwater depth = 1.9 m (for Q_{100})

Allowable velocity = 10 m/sn

Design Computations:

♦ **Determination of a trial size:**

By using Talbot Formula;

$A = C\sqrt[4]{M^3}$ in British units

or

$A = 5.79 * C\sqrt[4]{M^3}$ in metric (SI) units

C = 2/3 for rough hilly country of moderate slopes

and M = 0.521 km^2 = ~128.77 acres (1 acre ≅ 4046 * 10^{-6} km^2)

$$\Rightarrow A = \frac{2}{3} * \sqrt[4]{128.77^3} \cong 25.5 ft^2 \cong 2.37 m^2$$

or

$$A = 5.79 * \frac{2}{3} * \sqrt[4]{0.521^3} \cong 2.37 m^2$$

⇒ Select a trial size of 1.5 x 2.0 box culvert.

♦ **Inlet Control:**

In SI units;

Q_{10} = 5.32 m^3/sn and Q_{100} = 7.68 m^3/sn

By using the corresponding Inlet Control nomograph (Figure A.10);

Q_{10}/b_w = 5.32/1.5 ≅ 3.55 m^3/sn/m ⇒ HW/D ≅ 0.82 ⇒ HW ≅ 1.64 m.

and

Q_{100}/b_w = 7.68/1.5 ≅ 5.12 m^3/sn/m ⇒ HW/D ≅ 1.03 ⇒ HW ≅ 2.06 m.

or in British units (Figure A.5);

Q_{10} = 187.8 ft^3/sn (1 foot ≅ 0.3048 m)

Q_{10}/b_w = 187.8/4.92 ≅ 38.2 ft^3/sn/ft ⇒ HW/D ≅ 0.82 ⇒ HW ≅ 5.38 ft ≅ 1.64 m.

and

Q_{100} = 271.2 ft^3/sn

Q_{100}/b_w = 271.2/4.92 ≅ 55.1 ft^3/sn/ft ⇒ HW/D ≅ 1.03 ⇒ HW ≅ 6.76 ft ≅ 2.06 m.

Allowable Headwater Depth = 0.90 * 2.00 = 1.80 m for T = 10 years

and

Allowable Headwater Depth = min (3 * 2.00 ; 4.50) = 4.50 m for T = 100 years.

As can be seen

$HW_{10} = 1.64$ m $< HW_{all.} = 1.80$ m OK√

and

$HW_{100} = 2.06$ m $< HW_{all.} = 4.50$ m. OK√

- **Outlet Control:**

Slope of culvert = (139.22 – 138.00)/61 = 0.02

$k_e = 0.20$ (for 45° wingwall, grooved edge)

Critical depth;

$d_c = [q^2/g]^{1/3} = [(5.32/1.5)^2/9.81]^{1/3} \cong 1.09$ m for Q_{10}

and $d_c = [q^2/g]^{1/3} = [(7.68/1.5)^2/9.81]^{1/3} \cong 1.39$ m for Q_{100}

As TW = 1.6 m < D = 2.0 m for Q_{10};

$h_0 = \max [\ TW\ ;\ (d_c + D)/2] = \max [\ 1.6\ ;\ 1.54] = 1.6$ m.

By using the corresponding Outlet Control nomograph(Figure A.7) for L =200 ft,

$k_e = 0.2$, $A \cong 32.3$ ft^2 and $Q_{10} = 187.8$ ft^3/sn;

$H \cong 1.1$ ft $\cong 0.335$ m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.335 + 1.6 - 61*0.02 \cong 0.715$ m.

As TW = 1.9 m < D = 2.0 m for Q_{100};

$h_0 = \max [\ TW\ ;\ (d_c + D)/2] = \max [\ 1.9\ ;\ 1.70] = 1.9$ m.

By using the corresponding Outlet Control nomograph(Figure A.7)for L =200 ft,

$k_e = 0.2$, $A \cong 32.3$ ft^2 and $Q_{100} = 271.2$ ft^3/sn;

$H \cong 2.2$ ft $\cong 0.670$ m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.670 + 1.9 - 61*0.02 \cong 1.35$ m.

As can be seen INLET CONTROL governs both for Q_{10} and Q_{100} discharges.

Thus,

HW (for Q_{10}) = 1.64 m < $HW_{all.}$ = 1.80 m OK√

HW (for Q_{100}) = 2.04 m < $HW_{all.}$ = 4.50 m. OK√

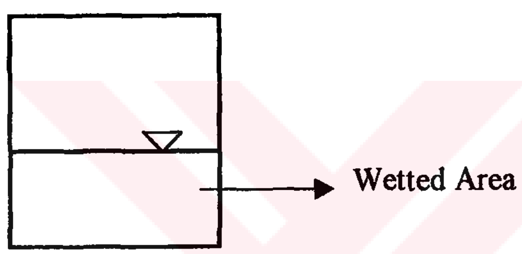
- **Velocity Limitation:**

a) For Q_{10}; d_n = 0.66 m & d_c = 1.09 m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R (hydraulic radius) = A/U (wetted area/wetted perimeter) such that



$\Rightarrow R = (1.5*0.66)/(1.5+2*0.66) \cong 0.351$ m

and S_0 = 0.02 (see Table B1 in Appendix B)

$\Rightarrow V = (1/0.013).(0.351)^{2/3}.(0.02)^{1/2} \cong 5.41$ m/sn.

b) For Q_{100}; d_n = 0.86 m & d_c = 1.39 m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R = A/U = (1.5*0.86)/(1.5+2*0.86) ≅ 0.401 m

and S_0 = 0.02

$\Rightarrow V = (1/0.013).(0.401)^{2/3}.(0.02)^{1/2} \cong 5.92$ m/sn.

V (for Q_{10}) = 5.41 m/sn. < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

V (for Q_{100}) = 5.92 m/sn < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

6.2 An Illustrative Example For A Single-Barrel Circular Culvert

❖ *Aydın – Denizli Highway* Aydın – Kuyucak section

Station-Km: 33 + 685

Given:

Drainage Area : 0.123 km^2

Invert upstream : 137.22 m

Invert downstream : 136.00 m

Height of Fill : 4.5 m

Length of Culvert : 61 m (~200 ft)

Terrain Type : A rough hilly country of moderate slopes

End Treatment : Headwall with square edge

$Q_{10} = 1.58\ m^3/sn = 55.8\ ft^3/sn$ Tailwater depth = 1.12 m (for Q_{10})

$Q_{100} = 2.31\ m^3/sn = 81.6\ ft^3/sn$ Tailwater depth = 1.22 m (for Q_{100})

Design Computations:

♦ **Determination of a trial size:**

By using Talbot Formula;

C = 2/3 for rough hilly country of moderate slopes

and M = 0.123 km^2 = ~30.40 acres

$$\Rightarrow A = \frac{2}{3} * \sqrt[4]{30.4^3} \cong 8.63 ft^2 \cong 0.80 m^2$$

or

$$A = 5.79 * \frac{2}{3} * \sqrt[4]{0.123^3} \cong 0.80 m^2$$

⇒ Select a trial size of D = 1.2 m circular culvert.

- **Inlet Control:**

In SI units;

$Q_{10} = 1.58\ m^3/sn$ and $Q_{100} = 2.31\ m^3/sn$

By using the corresponding Inlet Control nomograph (Figure A.11);

$Q_{10} = 1.58\ m^3/sn\ \&\ D = 1.2\ m \Rightarrow HW/D \cong 0.87 \Rightarrow HW \cong 1.04\ m.$

and

$Q_{100} = 2.31\ m^3/sn\ \&\ D = 1.2\ m \Rightarrow HW/D \cong 1.18 \Rightarrow HW \cong 1.42\ m.$

or in British units (Figure A.6,b);

$Q_{10} = 55.8\ ft^3/sn\ \&\ D \cong 47.2\ in \Rightarrow HW/D \cong 0.87 \Rightarrow HW \cong 41.06\ in \cong 1.04\ m.$

and

$Q_{100} = 81.6\ ft^3/sn\ \&\ D \cong 47.2\ in \Rightarrow HW/D \cong 1.18 \Rightarrow HW \cong 55.70\ in \cong 1.42\ m.$

Allowable Headwater Depth = 0.90 * 1.20 = 1.08 m for T = 10 years

and

Allowable Headwater Depth = min (3 * 1.2 ; 4.50) = 3.60 m for T = 100 years.

As can be seen

$HW_{10} = 1.04\ m < HW_{all.} = 1.08\ m$ OK√

and

$HW_{100} = 1.42\ m < HW_{all.} = 3.60\ m.$ OK√

- **Outlet Control:**

Slope of culvert = (137.22 – 136.00)/61 = 0.02

$k_e = 0.50$ (for square edge w/headwall)

Critical depth;

$d_c \cong 0.69\ m$ for Q_{10}

and $d_c \cong 0.84\ m$ for Q_{100}

As $TW = 1.12$ m $< D = 1.2$ m for Q_{10};

$h_0 = \max [\ TW\ ; (d_c + D)/2] = \max [\ 1.12\ ; 0.94] = 1.12$ m.

By using the corresponding Outlet Control nomograph(Figure A.8) for L =200 ft,

$k_e = 0.5$, $D = 47.2$ in and $Q_{10} = 55.8$ ft^3/sn;

$H \cong 0.69$ ft $\cong 0.21$ m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.21 + 1.12 - 61*0.02 \cong 0.11$ m.

As $TW = 1.22$ m $> D = 1.2$ m for Q_{100};

$h_0 = TW = 1.22$ m.

By using the corresponding Outlet Control nomograph(Figure A.8) for L =200 ft,

$k_e = 0.2$, $D = 47.2$ in and $Q_{100} = 81.6$ ft^3/sn;

$H \cong 1.76$ ft $\cong 0.54$ m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.54 + 1.22 - 61*0.02 \cong 0.54$ m.

As can be seen INLET CONTROL governs both for Q_{10} and Q_{100} discharges.

Thus,

HW (for Q_{10}) = 1.04 m $< HW_{all.} = 1.08$ m   OK√

HW (for Q_{100}) = 1.42 m $< HW_{all.} = 3.60$ m.   OK√

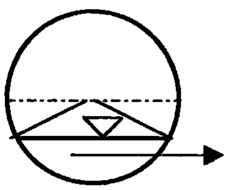
- **Velocity Limitation:**

a) For Q_{10}; $d_n = 0.44$ m & $d_c = 0.69$ m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R = A/U such that



→ Wetted Area = (Area of Circle Segment) – (Triangular Area)

Area of Circle Segment = $(\alpha/360°).\pi.R^2 = (149°/360°).\pi.(0.6)^2 = 0.47\ m^2$

Area of Triangle = $(1/2).R^2.\sin\alpha = (1/2).(0.6)^2.\sin(149°) = 0.09\ m^2$

$\Rightarrow$ Wetted Area = $0.47 - 0.09 = 0.38\ m^2$.

and Wetted Perimeter = $(\alpha/360°).2.\pi.R = (149°/360°).2.\pi.(0.6) = 1.56$ m.

$\Rightarrow$ Hydraulic Radius, $R = A/U = 0.38/1.56 = 0.24$ m

and $S_0 = 0.02$

$\Rightarrow V = (1/0.013).(0.24)^{2/3}.(0.02)^{1/2} \cong 4.20$ m/sn.

b) For Q_{100}; $d_n = 0.54$ m & $d_c = 0.84$ m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where $R = A/U = 0.49/1.76 = 0.28$ m

and $S_0 = 0.02$

$\Rightarrow V = (1/0.013).(0.28)^{2/3}.(0.02)^{1/2} \cong 4.66$ m/sn.

V (for Q_{10}) = 4.20 m/sn. < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

V (for Q_{100}) = 4.66 m/sn < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

6.3 An Illustrative Example For A Multiple-Barrel Box Culvert

❖ *Aydın – Denizli Highway* Aydın – Kuyucak section

Station-Km: 34 + 148

Given:

Drainage Area	: 2.17 km^2
Invert upstream	: 136.22 m
Invert downstream	: 135.00 m
Height of Fill	: 4.5 m
Length of Culvert	: 61 m (~200 ft)
Terrain Type	: A rough hilly country of moderate slopes
End Treatment	: 45° wingwall with grooved edge

Q_{10} = 12.5 m^3/sn = 441.4 ft^3/sn Tailwater depth = 2.5 m (for Q_{10})

Q_{100} = 18.7 m^3/sn = 660.4 ft^3/sn Tailwater depth = 3.3 m (for Q_{100})

Design Computations:

♦ **Determination of a trial size:**

By using Talbot Formula;

C = 2/3 for rough hilly country of moderate slopes

and M = 2.17 km^2 = ~536.33 acres

$$\Rightarrow A = \frac{2}{3} * \sqrt[4]{536.33^3} \cong 74.3 ft^2 \cong 6.9 m^2$$

or

$$A = 5.79 * \frac{2}{3} * \sqrt[4]{2.17^3} \cong 6.9 m^2$$

⇒ Select a trial size of 2 x 1.5 x 2.0 box culverts.

- **Inlet Control:**

In SI units;

$Q_{10} = 12.5\ m^3/sn$ and $Q_{100} = 18.7\ m^3/sn$

For each single culvert barrel;

$Q_{10}/2 = 12.5/2 = 6.25\ m^3/sn$ and $Q_{100}/2 = 18.7/2 = 9.35\ m^3/sn$

By using the corresponding Inlet Control nomograph (Figure A.10);

$(Q_{10}/2)/b_W = 6.25/1.5 \cong 4.17\ m^3/sn/m \Rightarrow HW/D \cong 0.90 \Rightarrow HW \cong 1.80$ m.

and

$(Q_{100}/2)/b_W = 9.35/1.5 \cong 6.23\ m^3/sn/m \Rightarrow HW/D \cong 1.22 \Rightarrow HW \cong 2.44$ m.

or in British units (Figure A.5);

$Q_{10} = 441.4\ ft^3/sn$

$(Q_{10}/2)/b_W = 220.7/4.92 \cong 44.8\ ft^3/sn/ft \Rightarrow HW/D \cong 0.90 \Rightarrow HW \cong 5.90\ ft \cong 1.8m.$

and

$Q_{100} = 660.4\ ft^3/sn$

$(Q_{100}/2)/b_W = 330.2/4.92 \cong 67.1\ ft^3/sn/ft \Rightarrow HW/D \cong 1.22 \Rightarrow HW \cong 8.0ft \cong 2.44m.$

Allowable Headwater Depth = 0.90 * 2.00 = 1.80 m for T = 10 years

and

Allowable Headwater Depth = min (3 * 2.00 ; 4.50) = 4.50 m for T = 100 years.

As can be seen

$HW_{10} = 1.80\ m \leq HW_{all.} = 1.80\ m$ OK√

and

$HW_{100} = 2.44\ m < HW_{all.} = 4.50\ m.$ OK√

- **Outlet Control:**

Slope of culvert = (136.22 – 135.00)/61 = 0.02

$k_e = 0.20$ (for 45° wingwall, grooved edge)

Critical depth;

$d_c = [q^2/g]^{1/3} = [(6.25/1.5)^2/9.81]^{1/3} \cong 1.21$ m for $Q_{10}/2$

and $d_c = [q^2/g]^{1/3} = [(9.35/1.5)^2/9.81]^{1/3} \cong 1.58$ m for $Q_{100}/2$

As TW = 2.5 m > D = 2.0 m for $Q_{10}/2$;

$h_0 =$ TW = 2.5 m.

By using the corresponding Outlet Control nomograph(Figure A.7) for L =200 ft,

$k_e = 0.2$, A $\cong$ 32.3 ft^2 and $Q_{10}/2 = 220.7$ ft^3/sn;

H $\cong$ 1.36 ft $\cong$ 0.414 m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.414 + 2.5 - 61*0.02 \cong 1.7$ m.

As TW = 3.3 m > D = 2.0 m for $Q_{100}/2$;

$h_0 =$ TW = 3.3 m.

By using the corresponding Outlet Control nomograph(Figure A.7) for L =200 ft,

$k_e = 0.2$, A $\cong$ 32.3 ft^2 and $Q_{100}/2 = 330.2$ ft^3/sn;

H $\cong$ 2.94 ft $\cong$ 0.896 m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.896 + 3.3 - 61*0.02 \cong 2.98$ m.

As can be seen INLET CONTROL governs for Q_{10} and OUTLET CONTROL for Q_{100} discharge..

Thus,

HW (for Q_{10}) = 1.80 m $\leq HW_{all.}$ = 1.80 m OK√

HW (for Q_{100}) = 2.98 m < $HW_{all.}$ = 4.50 m. OK√

- **Velocity Limitation:**

a) For Q_{10}; $d_n = 0.74$ m & $d_c = 1.21$ m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R = A/U = (1.5*0.74)/(1.5+2*0.74) $\cong$ 0.372 m

and $S_0 = 0.02$

$\Rightarrow V = (1/0.013).(0.372)^{2/3}.(0.02)^{1/2} \cong 5.63$ m/sn.

b) For Q_{100}; d_n = 1.01 m; d_c = 1.58 m & TW = 3.3 m

As Outlet Control governs this time;

$V = Q/A_0$

where A_0 is the cross-sectional area of flow in the culvert barrel of the outlet.

$A_0 = \max[(1.58*1.50) ; (3.3*1.5)] = \max[2.37 ; 4.95] = 4.95\ m^2$

but A_0 cannot be greater than total area, 3.0 m^2.

$\Rightarrow A_0 = 3.0\ m^2$ and $V = Q/A_0 = 9.35/3.0 = 3.12$ m/sn.

V (for Q_{10}) = 5.63 m/sn. < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

V (for Q_{100}) = 3.12 m/sn < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

6.4 An Illustrative Example For A Multiple-Barrel Circular Culvert

- *Aydın – Denizli Highway* Aydın – Kuyucak section

  Station-Km: 33 + 430

Given:

Drainage Area : 0.521 km^2

Invert upstream : 139.22 m

Invert downstream : 138.00 m

Height of Fill : 4.5 m

Length of Culvert : 61 m (~200 ft)

Terrain Type : A rough hilly country of moderate slopes

End Treatment : Headwall with square edge

Q_{10} = 5.32 m^3/sn = 187.8 ft^3/sn Tailwater depth = 1.6 m (for Q_{10})

Q_{100} = 7.68 m^3/sn = 271.2 ft^3/sn Tailwater depth = 1.9 m (for Q_{100})

Design Computations:

- **Determination of a trial size:**

By using Talbot Formula;

C = 2/3 for rough hilly country of moderate slopes

and M = 0.521 km^2 = ~128.77 acres

$$\Rightarrow A = \frac{2}{3} * \sqrt[4]{128.77^3} \cong 25.5 ft^2 \cong 2.37 m^2$$

or

$$A = 5.79 * \frac{2}{3} * \sqrt[4]{0.521^3} \cong 2.37 m^2$$

⇒ Select a trial size of 2 x (D = 1.2 m) circular culverts.

♦ **Inlet Control:**

In SI units;

$Q_{10} = 5.32$ m^3/sn and $Q_{100} = 7.68$ m^3/sn

For each single culvert barrel;

$Q_{10}/2 = 5.32/2 = 2.66$ m^3/sn and $Q_{100}/2 = 7.68/2 = 3.84$ m^3/sn

By using the corresponding Inlet Control nomograph (Figure A.11);

$(Q_{10}/2) = 2.66$ m^3/sn & $D = 1.2$ m $\Rightarrow$ $HW/D \cong 1.30$ $\Rightarrow$ $HW \cong 1.56$ m.

and

$(Q_{100}/2) = 3.84$ m^3/sn & $D = 1.2$ m $\Rightarrow$ $HW/D \cong 1.9$ $\Rightarrow$ $HW \cong 2.28$ m.

or in British units (Figure A.6,b);

$(Q_{10}/2) = 93.9$ ft^3/sn & $D \cong 47.2$ in $\Rightarrow HW/D \cong 1.3 \Rightarrow HW \cong 61.36$ in $\cong 1.56$ m.

and

$(Q_{100}/2) = 135.6$ ft^3/sn & $D \cong 47.2$ in $\Rightarrow HW/D \cong 1.9 \Rightarrow HW \cong 89.68$ in $\cong 2.28$ m.

Allowable Headwater Depth = 0.90 * 1.20 = 1.08 m for T = 10 years

and

Allowable Headwater Depth = min (3 * 1.2 ; 4.50) = 3.60 m for T = 100 years.

As can be seen

$HW_{100} = 2.28$ m $< HW_{all.} = 3.60$ m. OK√

but

$HW_{10} = 1.56$ m $> HW_{all.} = 1.08$ m Not Acceptable!

Therefore, new culvert dimensions providing a larger opening should be selected

$\Rightarrow$ Select a trial size of standard 2 x (D = 60 in) circular culverts.

♦ **Inlet Control:**

In British units;

$Q_{10} = 187.8$ ft^3/sn and $Q_{100} = 271.2$ ft^3/sn

For each single culvert barrel;

$Q_{10}/2$ = 187.8 ft^3/sn /2 = 93.9 ft^3/sn and $Q_{100}/2$ = 271.2 ft^3/sn /2 = 135.6 ft^3/sn

By using the corresponding Inlet Control nomograph (Figure A.6,b);

$(Q_{10}/2)$ = 93.9 ft^3/sn & D = 60 in $\Rightarrow$ HW/D $\cong$ 0.82 $\Rightarrow$ HW $\cong$ 49.2 in $\cong$ 1.25 m.

and

$(Q_{100}/2)$ = 135.6 ft^3/sn & D = 60 in $\Rightarrow$ HW/D $\cong$ 1.03 $\Rightarrow$ HW $\cong$ 61.8 in $\cong$ 1.57 m.

Allowable Headwater Depth = 0.90 * 60 in = 54 in = 1.37 m for T = 10 years

and

Allowable Headwater Depth = min (3 * 1.524 ; 4.50) = 4.50 m for T = 100 years.

As can be seen

$HW_{10} = 1.25$ m $\leq HW_{all.} = 1.37$ m OK√

and

$HW_{100} = 1.57$ m $< HW_{all.} = 4.50$ m. OK√

- **Outlet Control:**

Slope of culvert = (139.22 – 138.00)/61 = 0.02

k_e = 0.50 (for square edge w/headwall)

Critical depth;

$d_c \cong 0.84$ m for Q_{10}

and $d_c \cong 1.02$ m for Q_{100}

As TW = 1.6 m > D = 1.52 m for Q_{10};

h_0 = TW = 1.6 m.

By using the corresponding Outlet Control nomograph(Figure A.8) for L =200 ft,

k_e = 0.5, D = 60 in and $(Q_{10}/2)$ = 93.9 ft^3/sn;

H $\cong$ 0.70 ft $\cong$ 0.21 m

$\Rightarrow HW = H + h_0 - L.S_0 = 0.21 + 1.6 - 61*0.02 \cong 0.59$ m.

As TW = 1.9 m > D = 1.52 m for Q_{100};

h_0 = TW = 1.9 m.

By using the corresponding Outlet Control nomograph(Figure A.8) for L =200 ft, $k_e = 0.2$, D = 60 in and $(Q_{100}/2) = 135.6\ ft^3/sn$;

$H \cong 1.2\ ft \cong 0.36\ m$

$\Rightarrow HW = H + h_0 - L.S_0 = 0.36 + 1.9 - 61*0.02 \cong 1.04\ m.$

As can be seen INLET CONTROL governs both for Q_{10} and Q_{100} discharges.

Thus,

HW (for Q_{10}) = 1.25 m < $HW_{all.}$ = 1.37 m. OK√

HW (for Q_{100}) = 1.57 m < $HW_{all.}$ = 4.50 m. OK√

- **Velocity Limitation:**

a) For Q_{10}; D = 1.52 m, d_n = 0.52 m & d_c = 0.84 m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R = A/U

Area of Circle Segment = $(\alpha/360°).\pi.R^2 = (143.2°/360°).\pi.(0.76)^2 = 0.72\ m^2$

Area of Triangle = $(1/2).R^2.\sin\alpha = (1/2).(0.76)^2.\sin(143.2°) = 0.17\ m^2$

$\Rightarrow$ Wetted Area = 0.72 – 0.17 = 0.55 m^2.

And Wetted Perimeter = $(\alpha/360°).2.\pi.R = (143.2°/360°).2.\pi.(0.76) = 1.90$ m.

$\Rightarrow$ Hydraulic Radius, R = A/U = 0.55/1.90 = 0.29 m

and $S_0 = 0.02$

$\Rightarrow V = (1/0.013).(0.29)^{2/3}.(0.02)^{1/2} \cong 4.77$ m/sn.

b) For Q_{100}; d_n = 0.64 m & d_c = 1.02 m

As Inlet Control governs, by using Manning's Formula;

$V = (1/n).R^{2/3}.S_0^{1/2}$

where R = A/U = 0.73/2.15 = 0.34 m

and $S_0 = 0.02$

$\Rightarrow V = (1/0.013).(0.34)^{2/3}.(0.02)^{1/2} \cong 5.30$ m/sn.

V (for Q_{10}) = 4.77 m/sn. < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

V (for Q_{100}) = 5.30 m/sn < $V_{all.}$ = 10 m/sn. OK√ (No Protection Required)

6.5 Computerized Solutions of Illustrative Examples

Numerous calculator and computer programs are available to aid in the design and analysis of highway culverts. In this thesis work, three computer programs on culvert analysis and design, namely CivilTools, Dodson HLW and CulvertMaster (by Haestad Methods, Inc.) have been examined, and computerized solutions to the above four examples have been obtained by using these programs in order to compare hand calculation solutions with the computer outputs (Appendix C). The major advantages of these programs over the traditional hand calculation method are:

- Increased accuracy over charts and nomographs.
- Rapid comparison of alternative sizes and inlet configurations.

However, it should be accepted that familiarity with culvert hydraulics and traditional methods of solution is necessary to provide a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities of hydraulic design computer programs.

6.5.1 Comparison of Program Outputs with Hand Calculations

When the solutions of the illustrative examples given in this chapter are compared with the outputs of the computer program packages given in Appendix C, the following evaluations can be made:

- CivilTools program software (CivilTools Civil Engineering Softwares) has given results almost the same as the hand computation method, for both single and multiple-barrel pipe culverts; however, there are slight differences between the computerized results and hand-solution results for either type of box culverts. This may be due to the precision in reading from corresponding nomographs, but the systematic behaviour of the difference rather than random indicates that it may be caused by a safety factor included in such computer programs.

- Dodson HLW computer program (Dodson & Associates, Inc.) gives almost the same results as the hand computation method again for either type of pipe culverts and gives results closer to the hand method also for both types of box culverts. By considering this observation, it may be concluded that Dodson HLW is a more reliable program than CivilTools. Indeed, the compatibility in the determination of governing condition for culverts, either inlet or outlet, between the proposed hand solution method and Dodson HLW program software seems to support this basic conclusion.

- Since the third program investigated, namely CulvertMaster (Haestad Methods, Inc.), does allow to select a trial culvert size among only certain given culvert dimensions in British units, a comparison can not be made and a conclusion can not be reached easily. However, this program may definitely be suggested for use in practice due to the easiness in getting results in a formal report supported by certain graphics.

CHAPTER SEVEN
CONCLUSIONS

The purpose of this study has been to investigate the characteristics of highway culverts and propose a complete culvert design procedure which can directly be utilized to solve most of the culvert engineering problems encountered in application. In order to examine the reliability of this proposed procedure, several examples selected from some real-world culvert locations on Aydın-Denizli Highway have been re-designed by using this procedure and the results have been evaluated and then compared with the solutions of three different culvert design package programs.

Culverts are hydraulic structures which are used to convey water through an embankment at locations where a highway and a river intersect each other. Although it seems that a bridge performs the same function, culverts principally differ from bridges especially when some of their basic characteristics have been considered (span length, flow conditions, etc.). In order for an efficient and economic operation of a culvert within a drainage system, it should be located properly so as to provide the stream with a direct entrance and a direct exit and to prevent the stream from changing its course near the ends of the culvert. Furthermore, the slope of the culvert should be as close as possible to the slope of the natural stream and any change in this slope along the culvert should be avoided unless otherwise is required by certain specific conditions. Culvert length, on the other hand, should be long enough to prevent the ends of culvert from clogging with sediment or spreading embankment.

In order to design a roadway culvert by using the methodology explained in the end of this thesis work, the required data should be gathered from field surveys and the design flood discharge, being also one of the required data for analysis and design, should be estimated by using certain discharge computation methods valid for different-sized drainage areas. Originally, the most difficult and problematic task of the culvert design process, misevaluation of which is also the most frequent cause of culvert failures, is the determination of this design discharge properly. As it would not be always possible to design a hydraulic structure with consideration of the most catastrophic event (i.e. zero risk condition) regarding the economic considerations, the design engineer should select an acceptable recurrence interval to be used in estimating the design discharge so that the culvert will have been designed to operate in a manner that is within acceptable limits of risk at that flow rate.

If the given alternative methodology for culvert design is examined, it can easily be seen that supercritical flow at the culvert outlet is not desired to avoid high velocities and thus scouring of the natural streambed. Indeed, a new tailwater depth above the critical depth is assumed and used in calculations instead of the exact tailwater depth so as to provide a subcritical flow at the outlet after the design operations and also some amount of safety factor is included in computations.

Having compared the hand-solution results with the outputs of computer softwares, there may not be realized any significant difference between them, even though there are some slight differences between the results of three different computer program packages. However, it should be admitted that the use of such programs would increase the accuracy in reading from related charts and nomographs and would bring in the advantage of a rapid comparison of alternative sizes and inlet configurations.

To conclude, it can be briefly stated that culvert design is a very difficult stage in culvert engineering; however, the use of available user-friendly computer programs have been decreasing this difficulty day by day and that will act as a continuous development in culvert engineering technology unless it is forgotten that engineering judgement is the most basic tool a design engineer needs rather than most computer programs.

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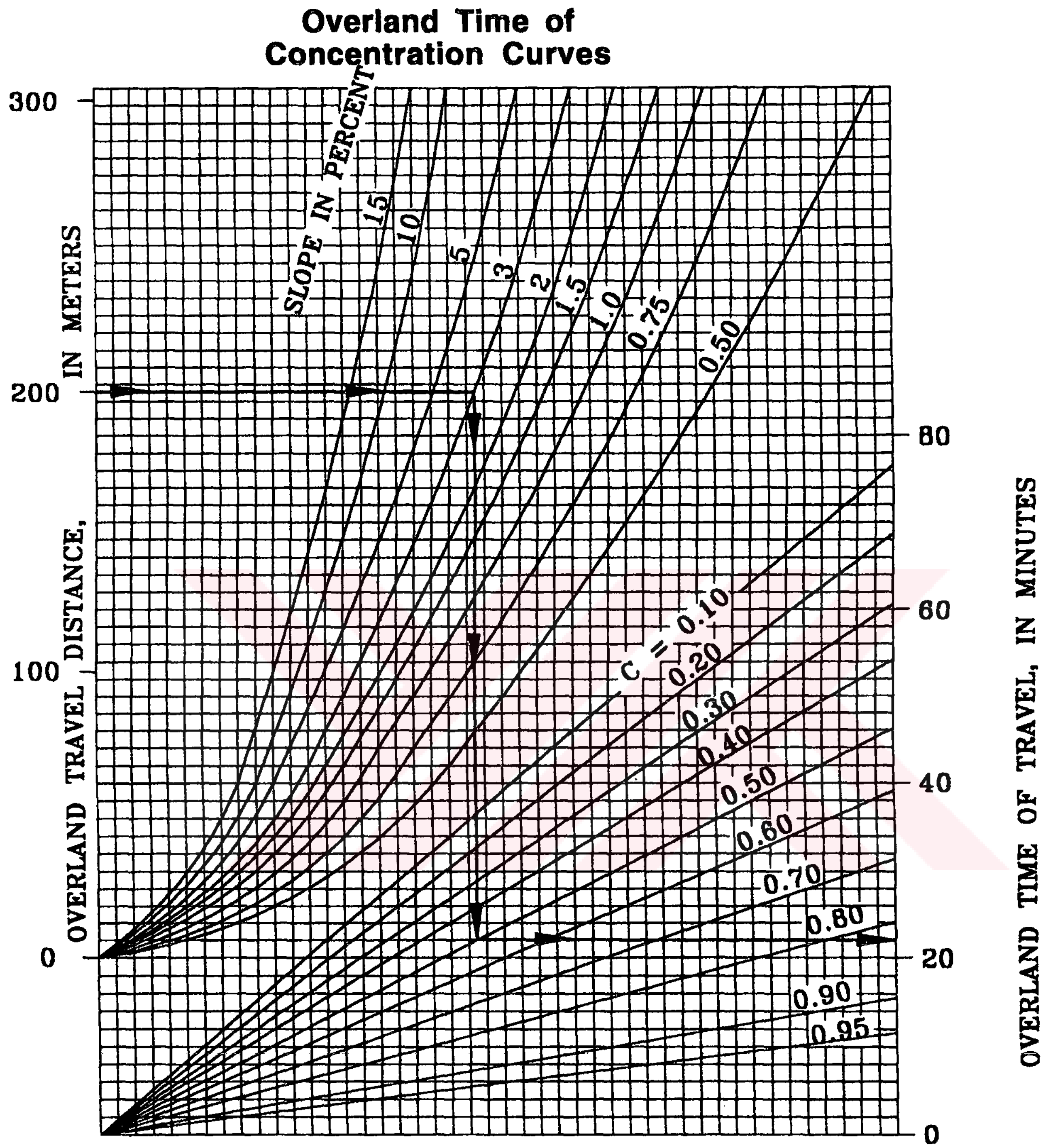
APPENDICES

APPENDIX A



$$T_o = \frac{3.3(1.1-C)(L)^{1/2}}{[S(100)]^{1/3}}$$

Where:
C = Runoff Coefficient
L = Overland Travel Distance in meters
S = Slope in m/m
T_o = Time in minutes

Figure A.1 Overland time of concentration curves (HDM, 1995, c.810)

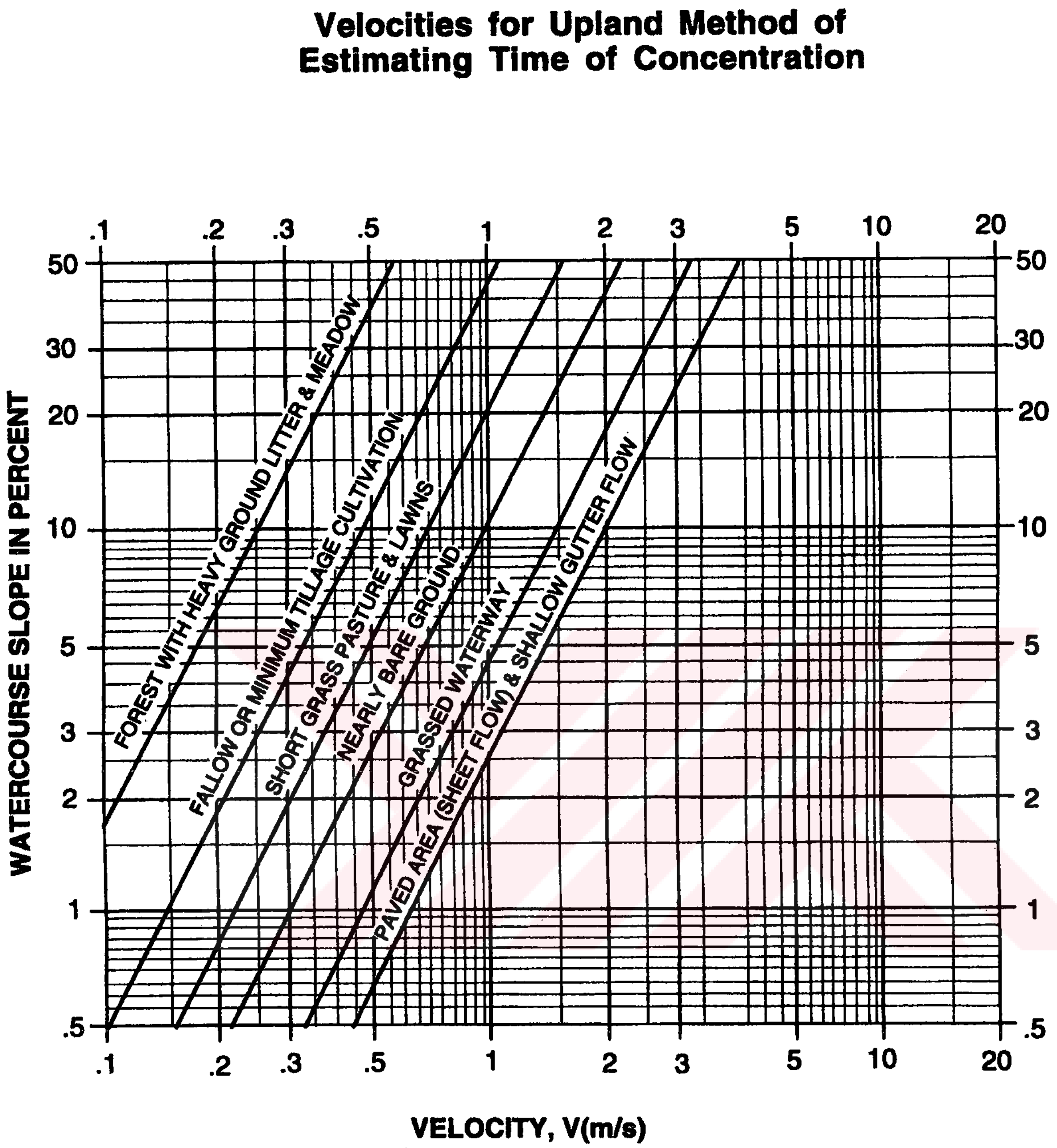


Figure A.2 Velocities for upland method of estimating time of concentration (HDM, 1995, c.810)

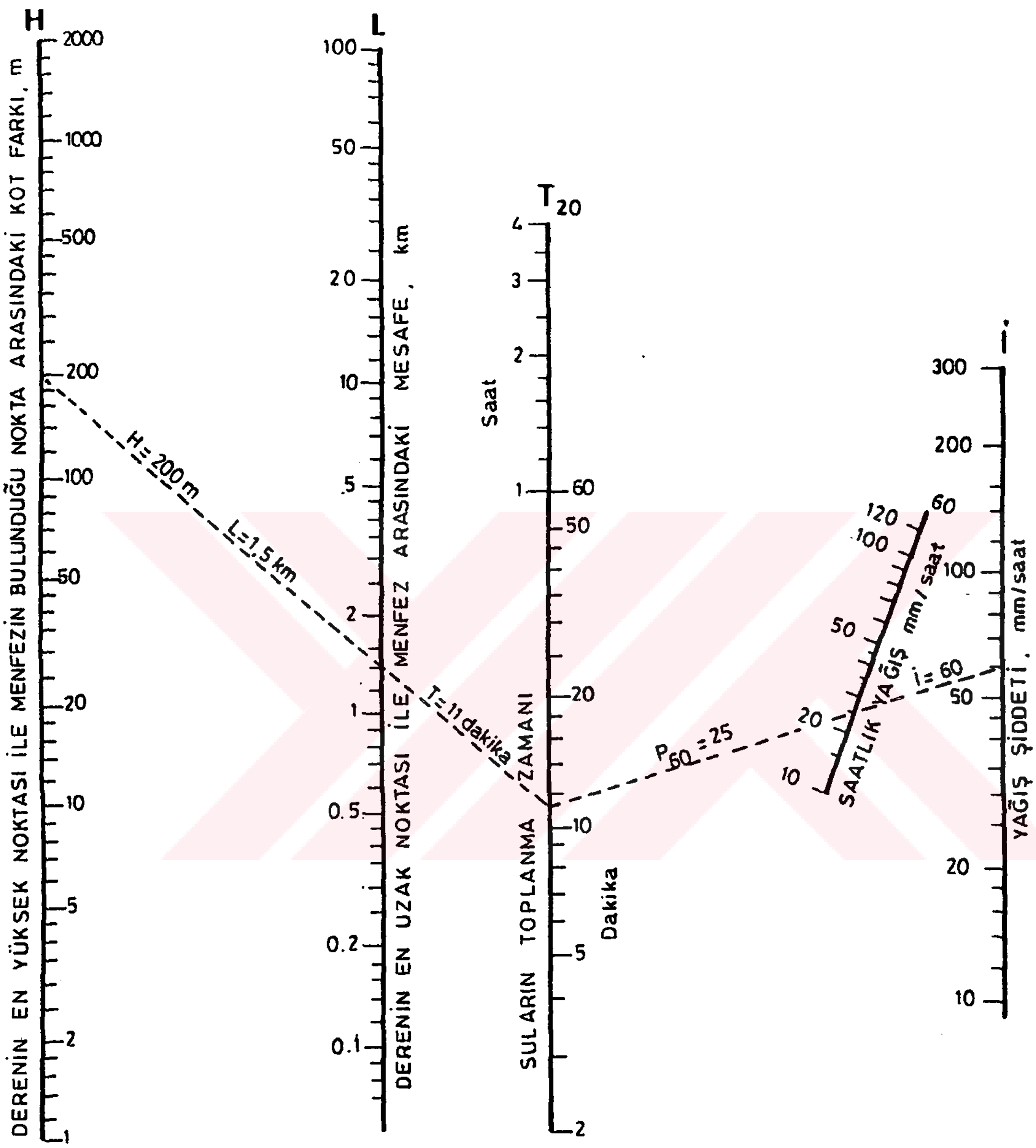
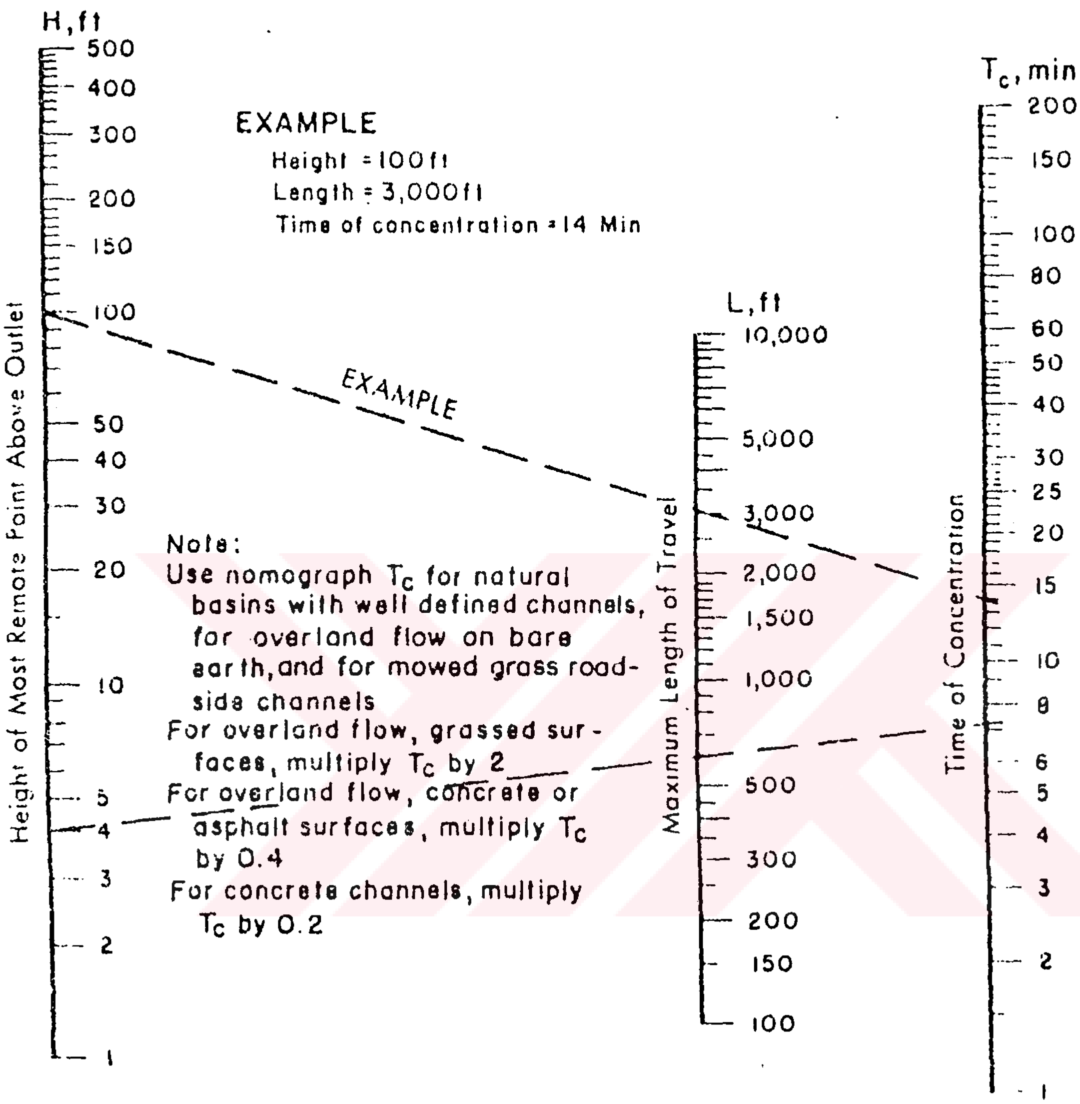


Figure A.3 Nomogram for the time of channel flow, t_2 (Atalay et al., 1973)



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No 6, June 1940, p. 362

Figure A.4 Time of concentration of rainfall on small drainage basins (AISI, 1983)

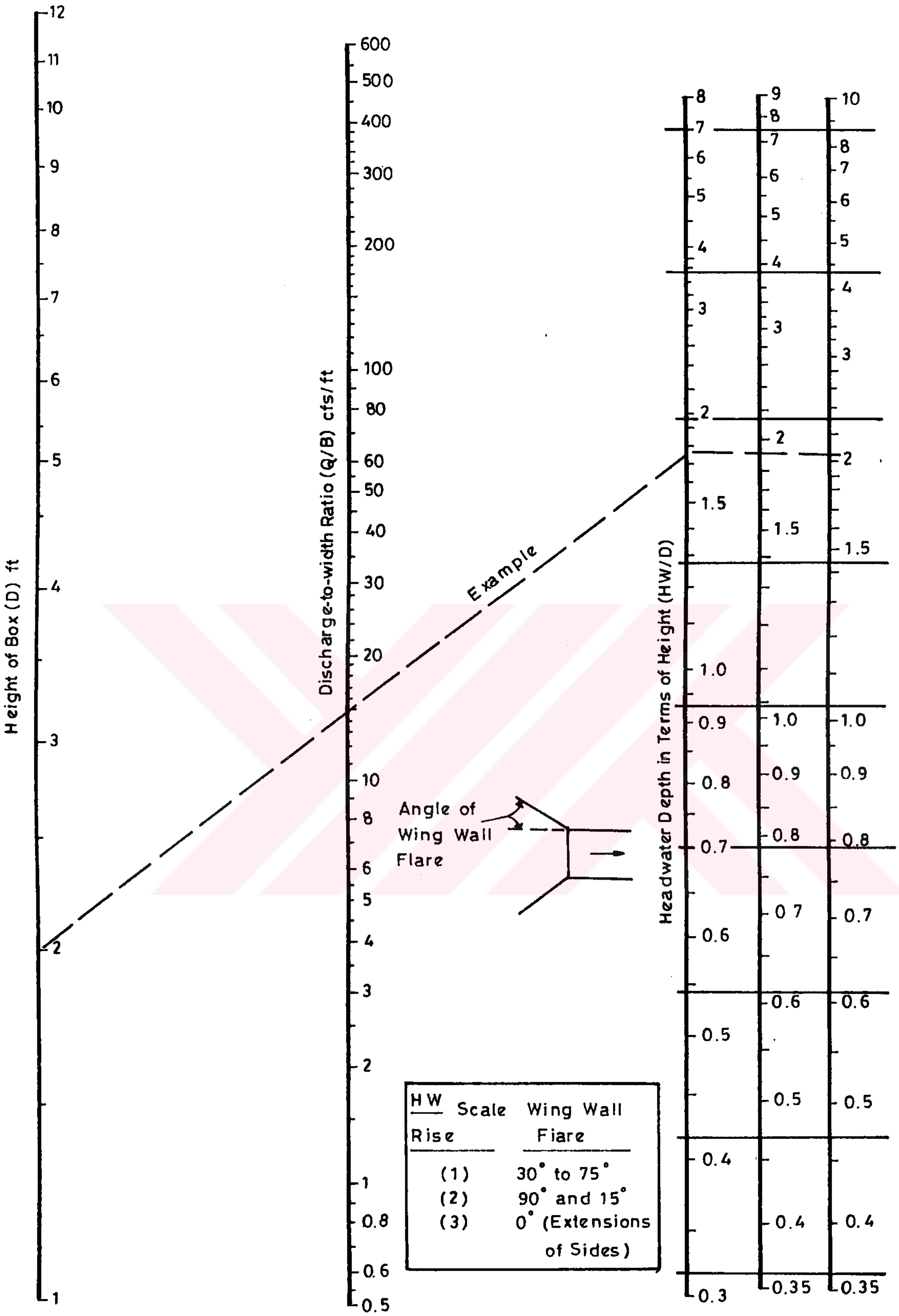


Figure A.5 Inlet control nomograph - box culverts (AISI, 1983)

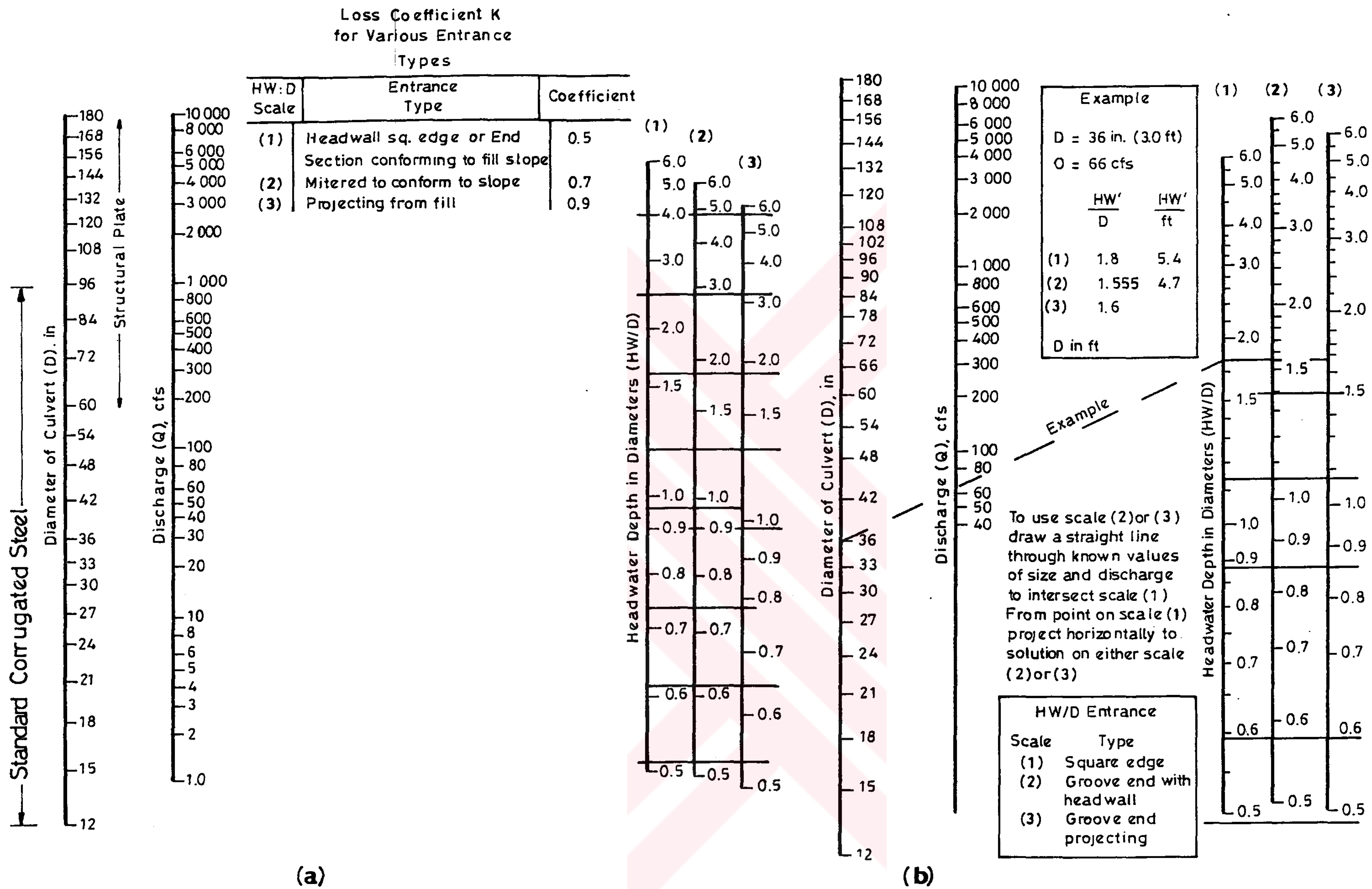


Figure A.6 Inlet control nomograph - circular pipe culverts (U.S. Bureau of Public Roads Division)

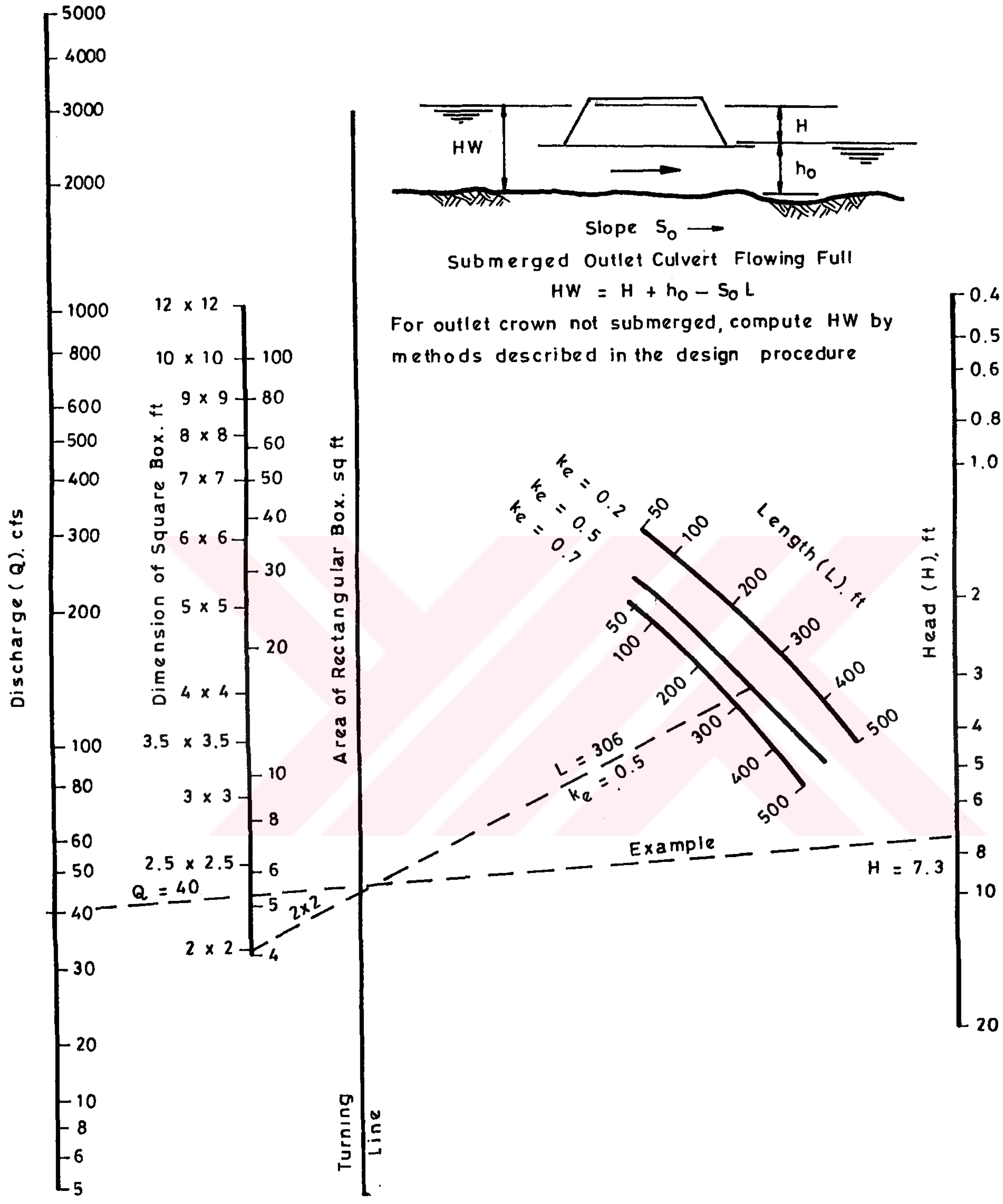


Figure A.7 Outlet control nomograph - box culverts (U.S. Dept. of Commerce)

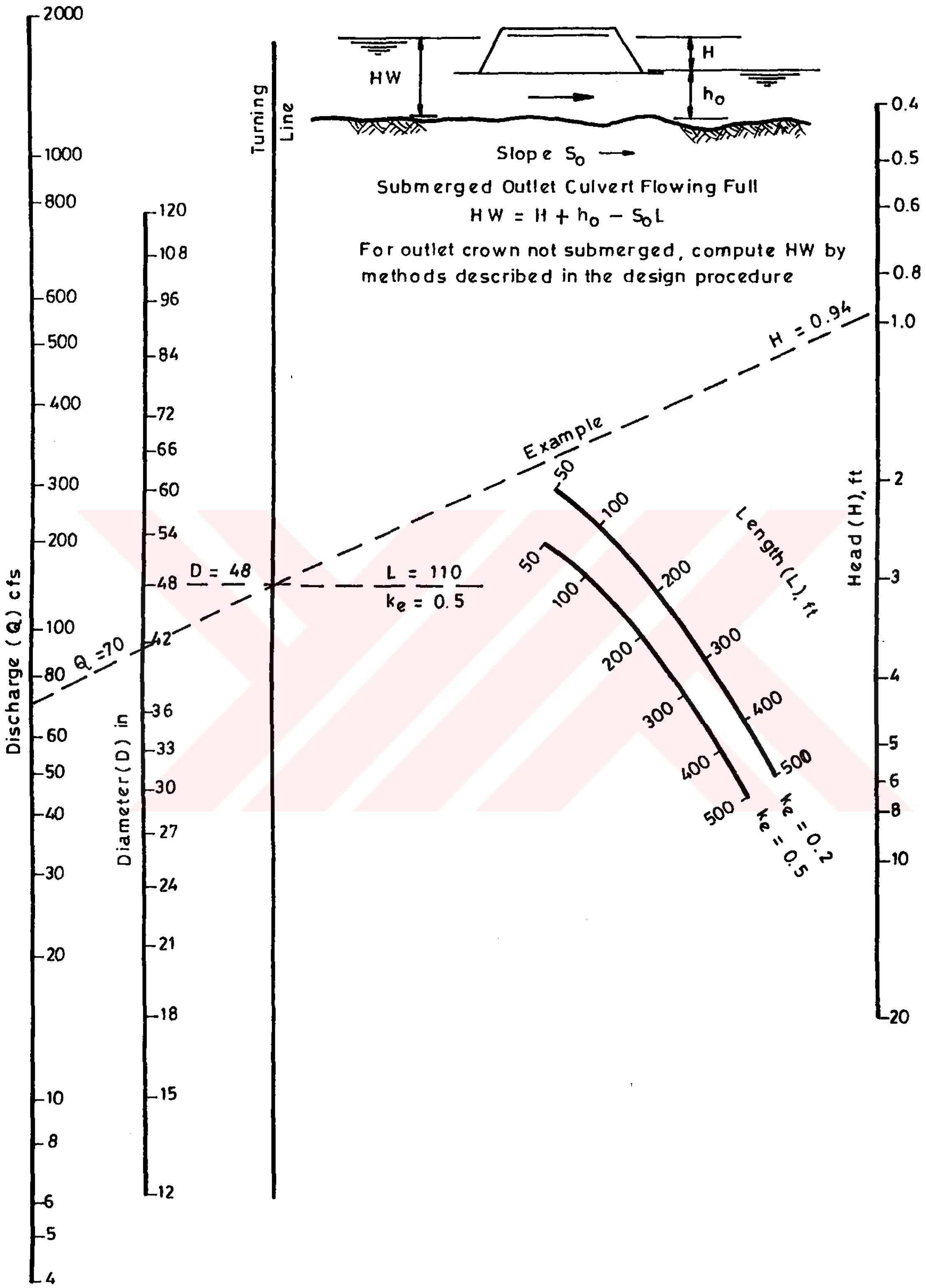


Figure A.8 Outlet control nomograph circular reinforced concrete pipe culverts (U.S. Dept. of Commerce)

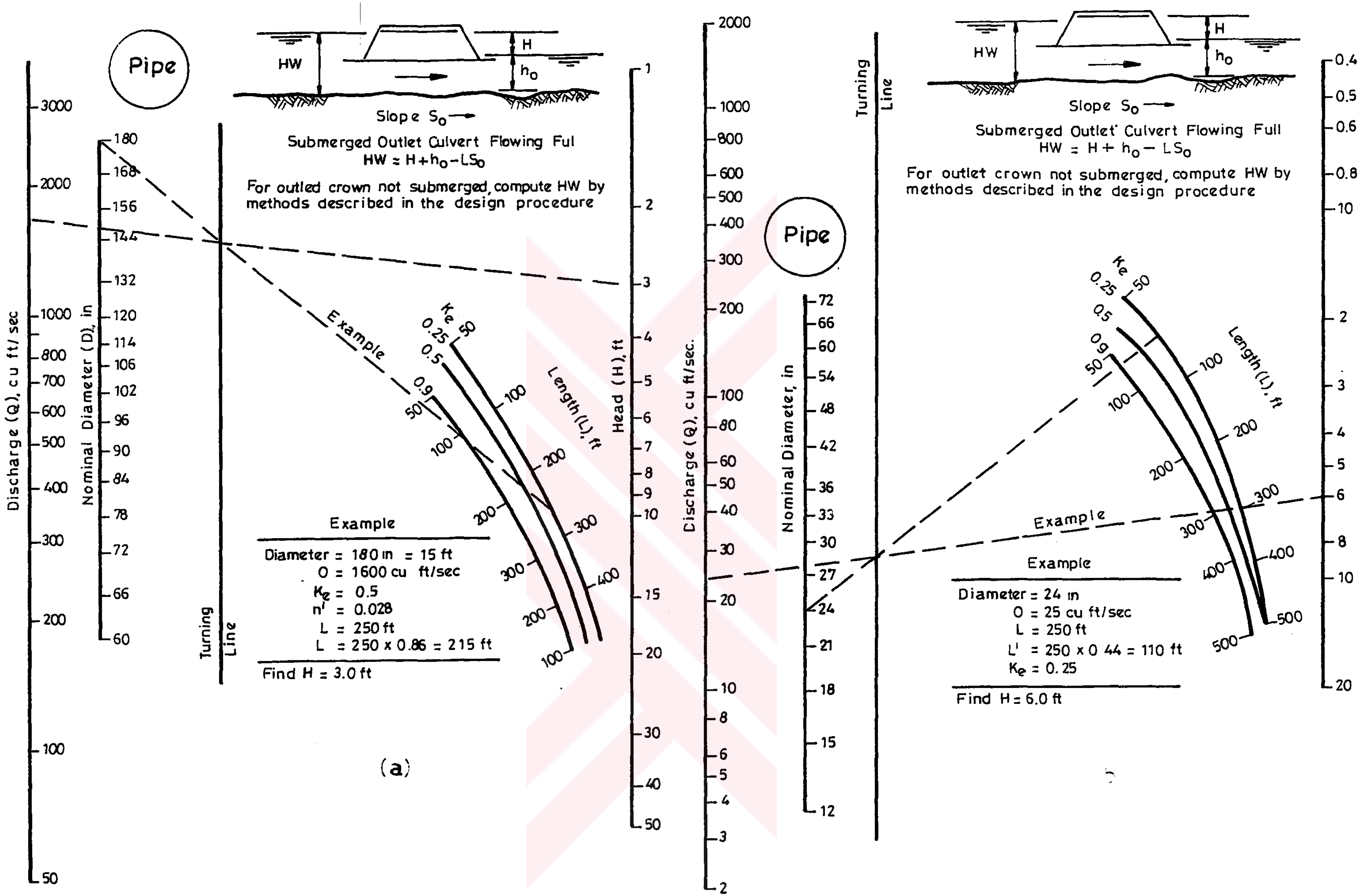


Figure A.9 Outlet control nomograph – (a) structural plate corrugated steel pipe culverts, (b) standard corrugated steel pipe culverts (AISI, 1983)

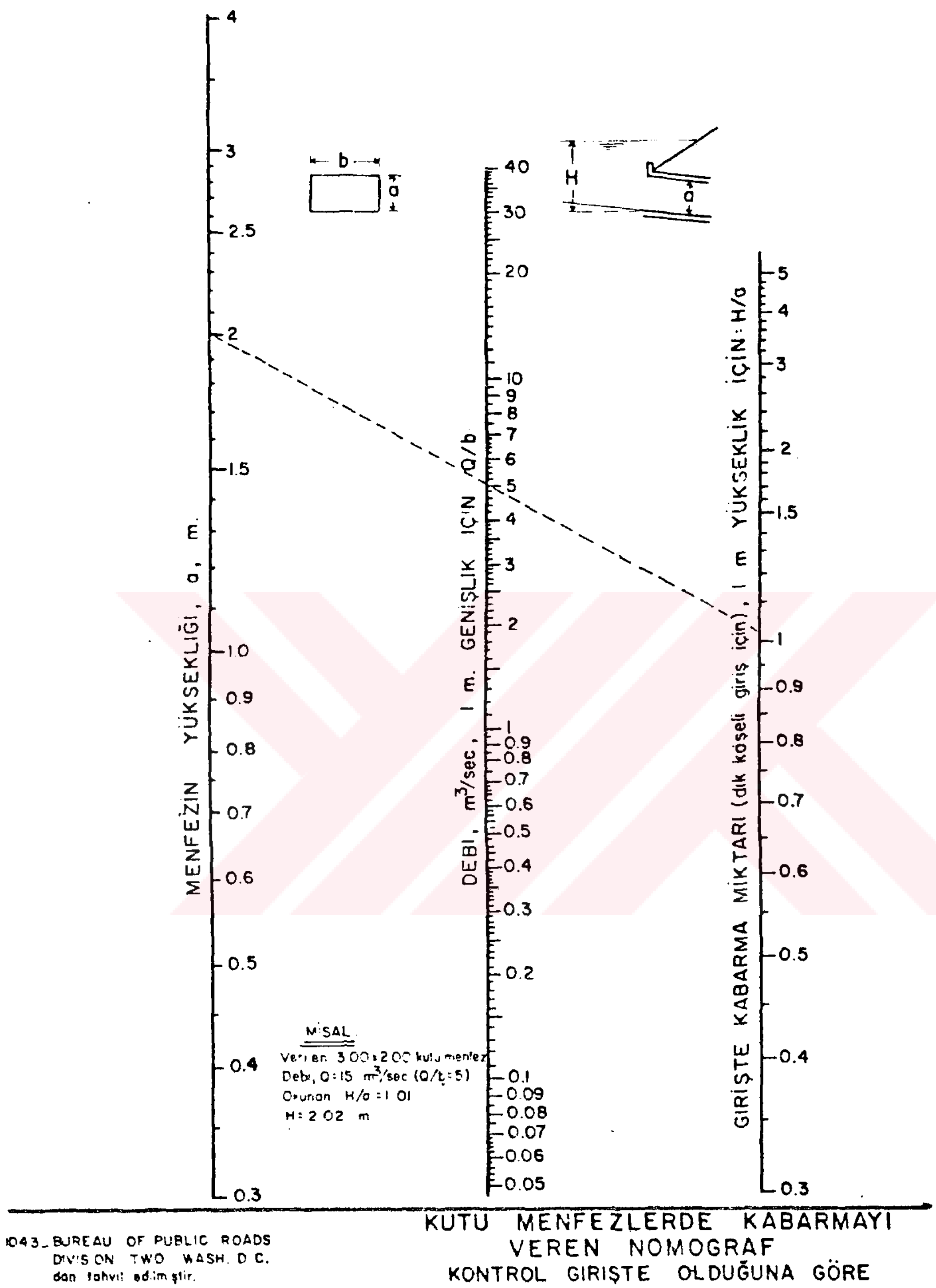
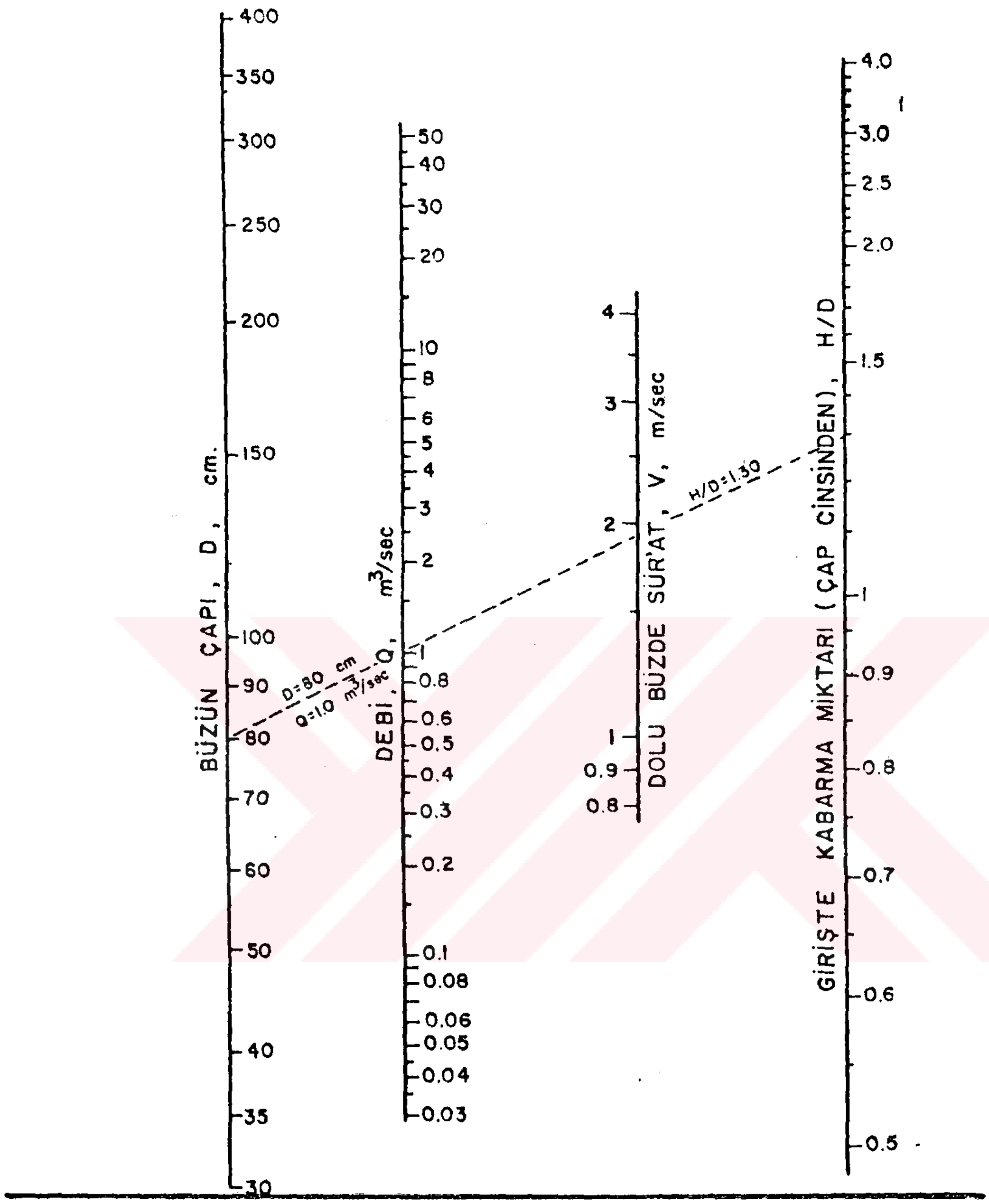


Figure A.10 Inlet control nomograph - box culverts (in SI units) (revised from the U.S. Bureau of Public Roads Division)



KISMEN DOLU AKAN BÜZLERDE
KABARMAYI VEREN NOMOGRAF
KONTROL GİRİŞTE OLDUĞUNA GÖRE

1042_U.S. PUBLIC ROADS ADMINISTRATION
DIVISION TWO WASHINGTON, D.C.
dan tahvil edilmiştir.

Figure A.11 Inlet control nomograph - circular pipe culverts flowing partly full (in SI units) (revised from the U.S. Public Roads Administration Division)

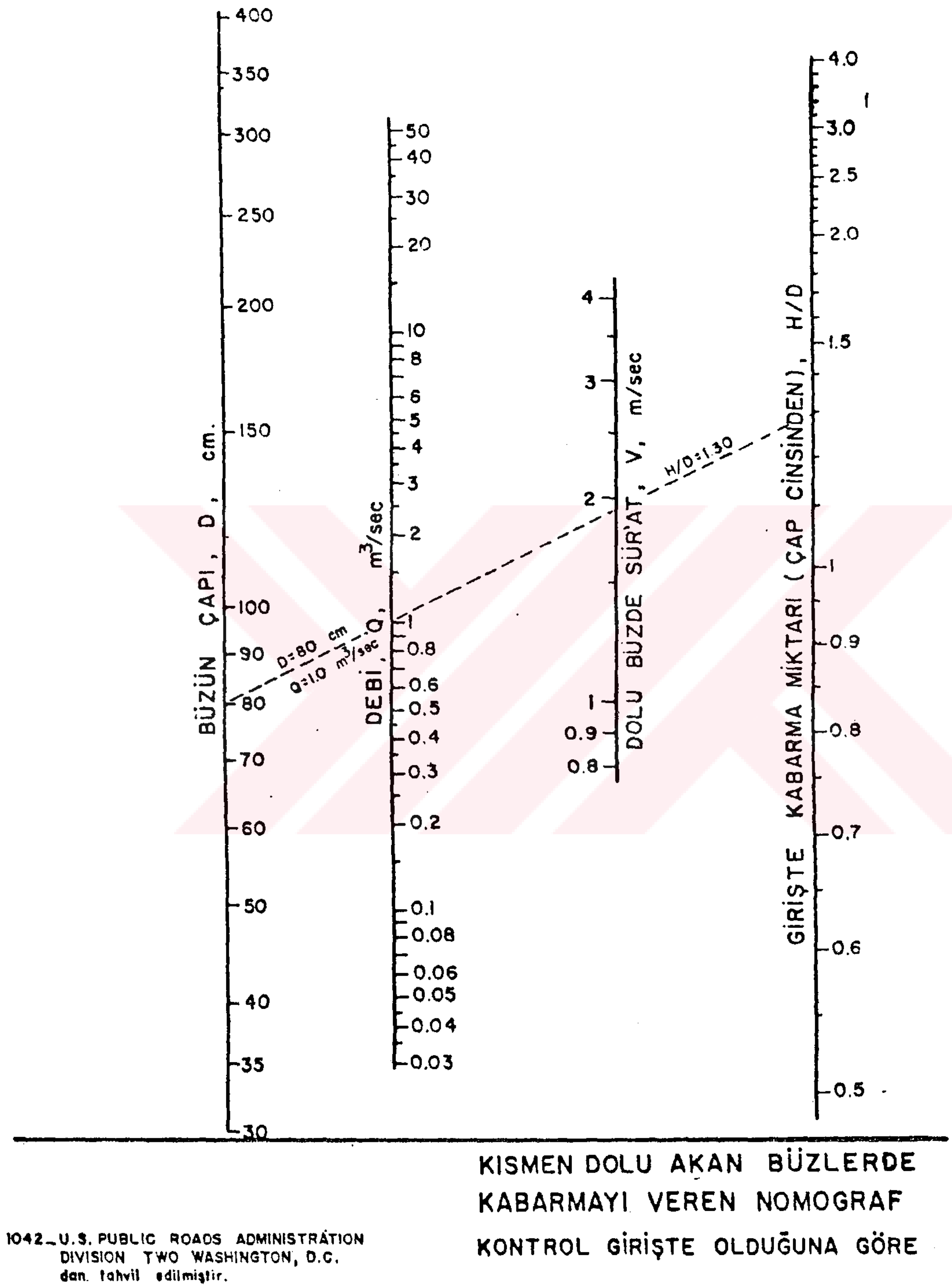


Figure A.11 Inlet control nomograph - circular pipe culverts flowing partly full (in SI units) (revised from the U.S. Public Roads Administration Division)

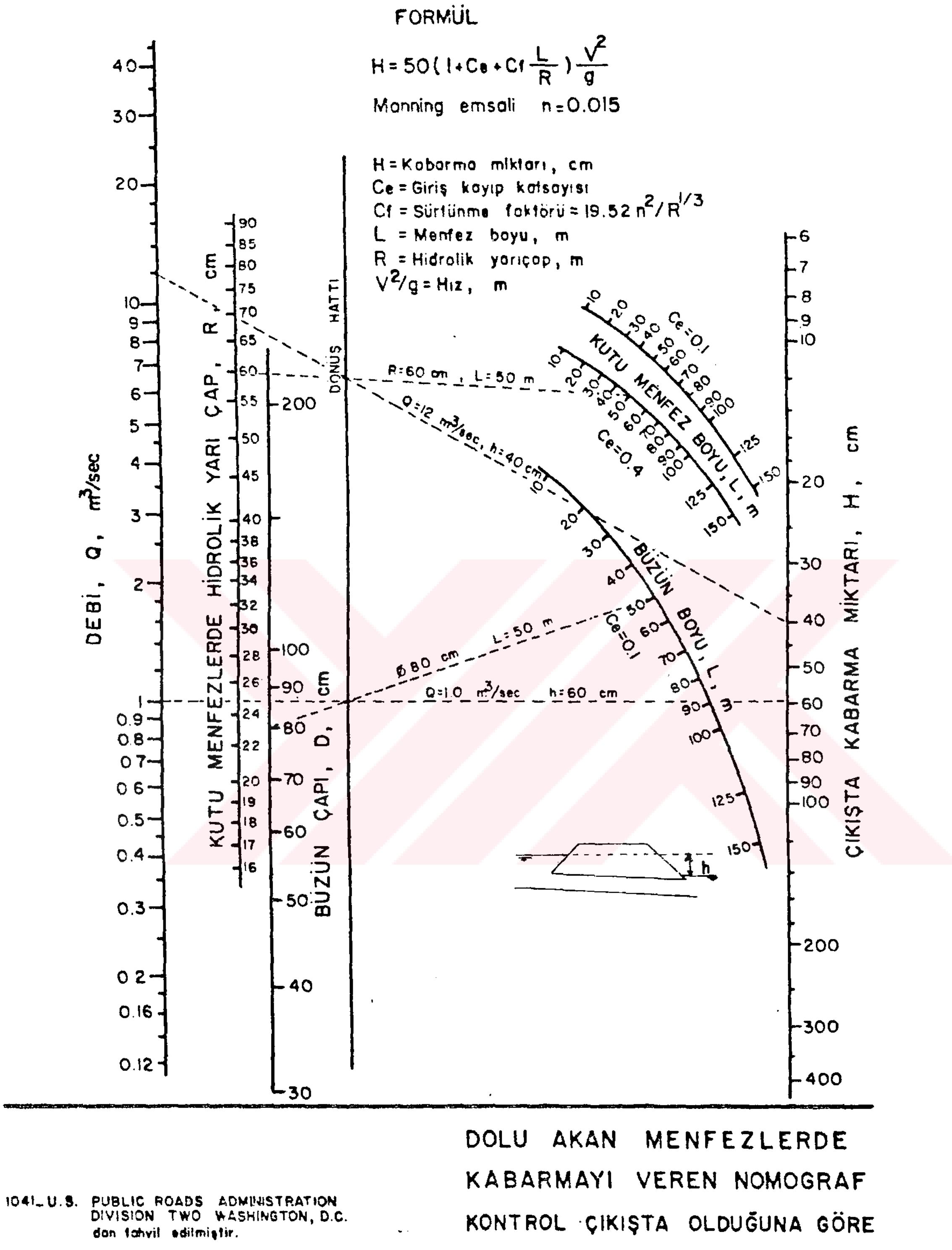


Figure A.12 Outlet control nomograph – box and circular pipe culverts flowing full (in SI units) (revised from the U.S. Public Roads Administration Division)

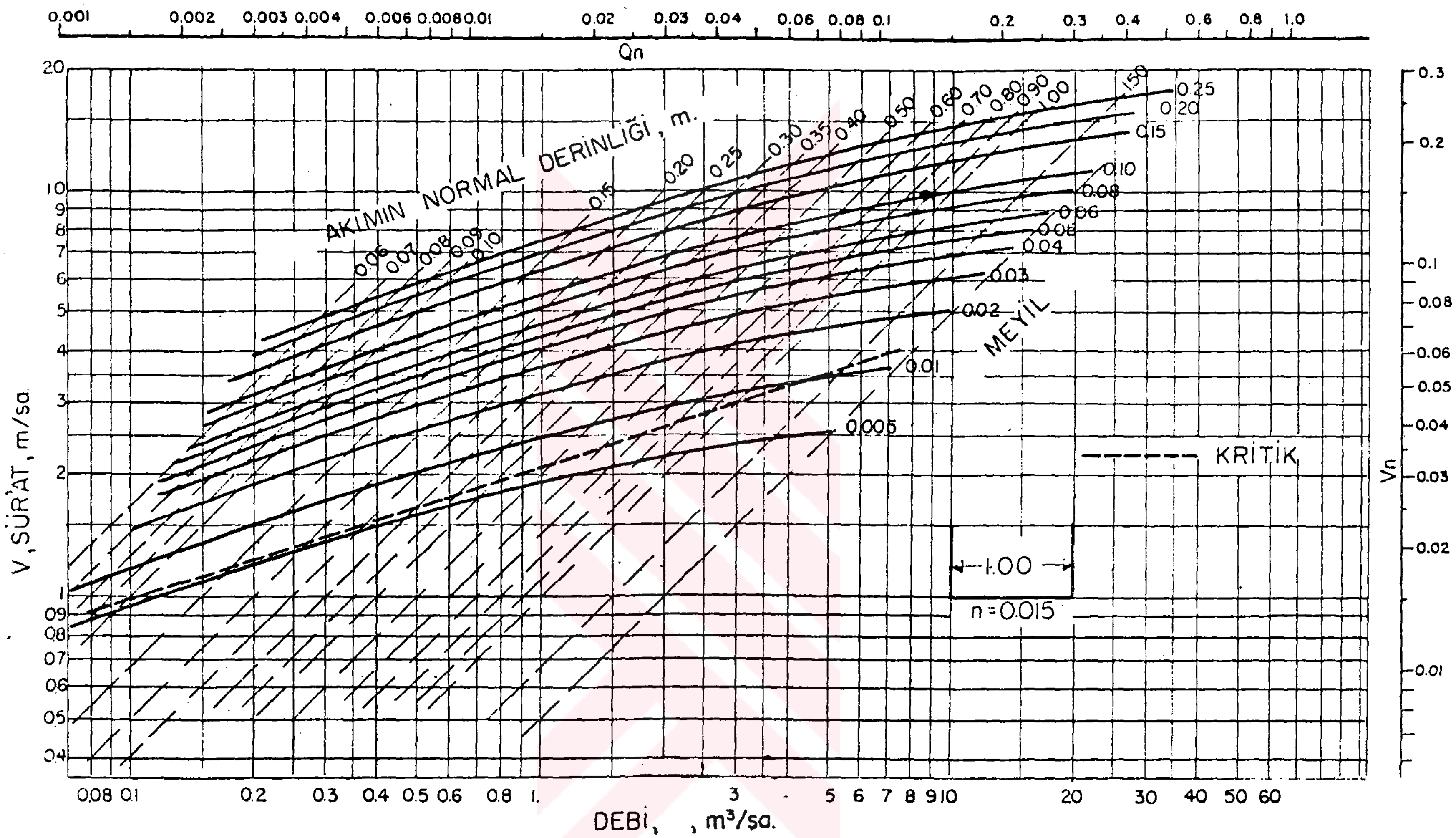


Figure A.13 Discharge in box culverts (for b = 1.00 m)

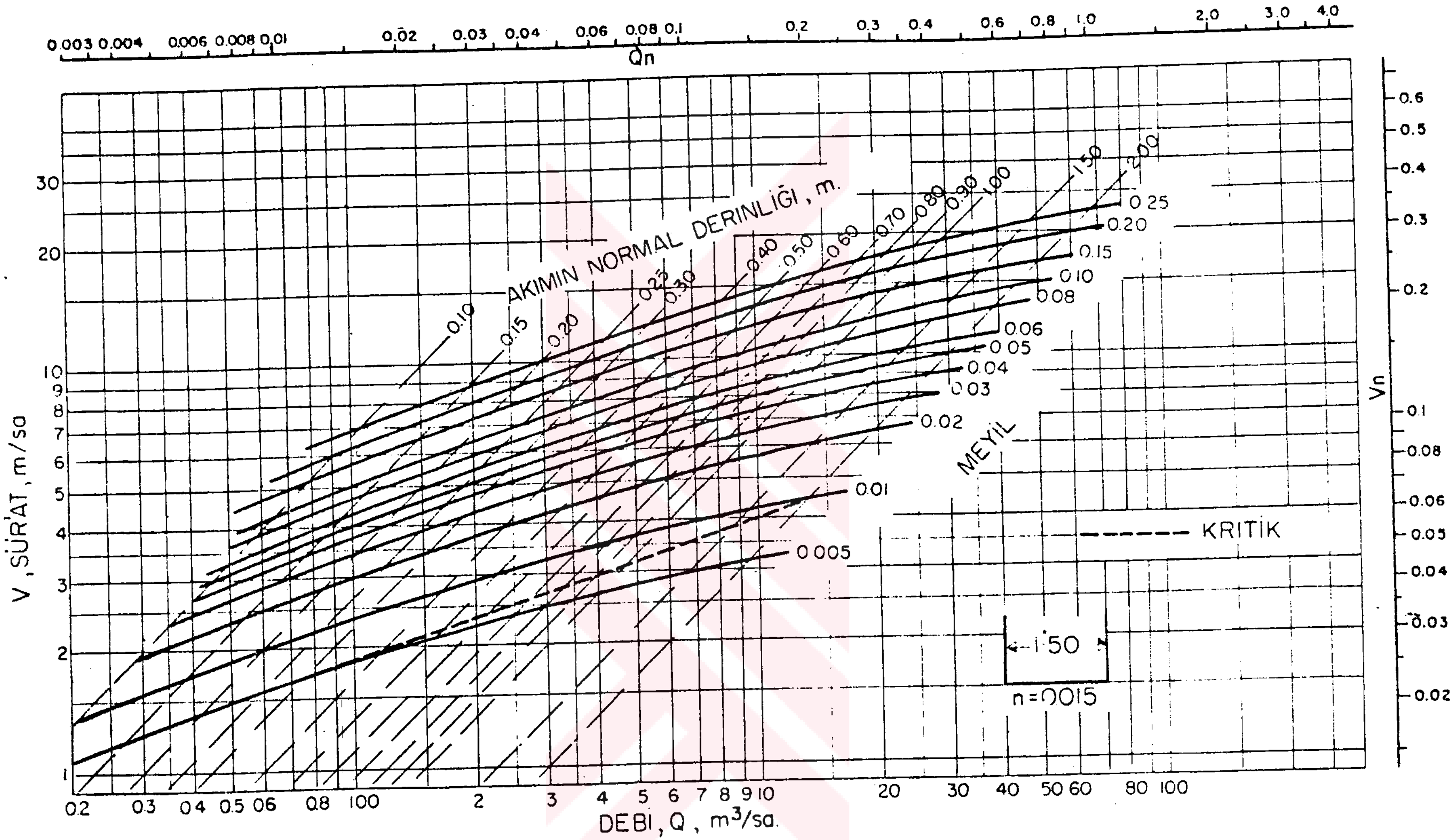


Figure A.14 Discharge in box culverts (for b = 1.50 m)

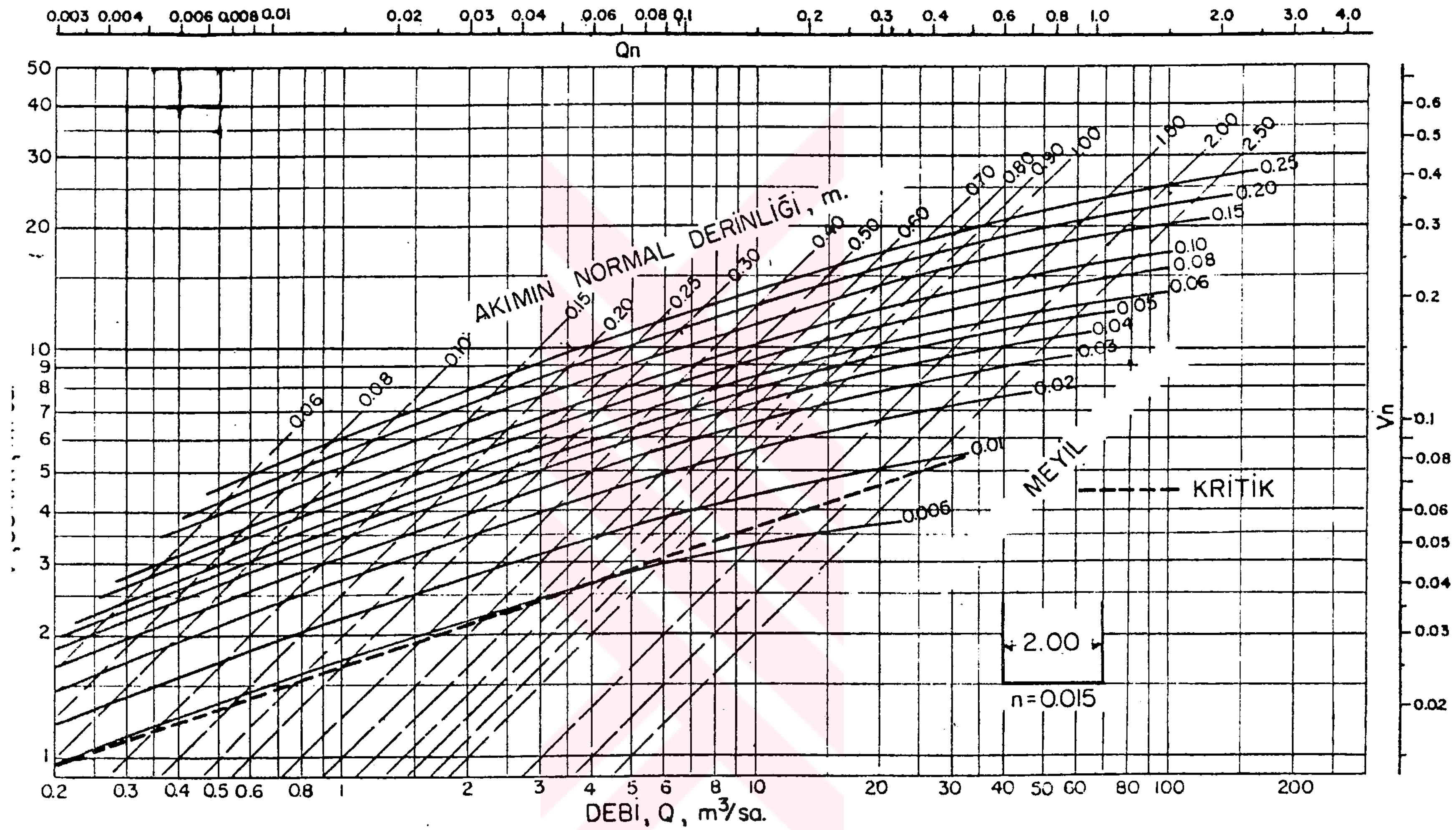


Figure A.15 Discharge in box culverts (for b = 2.00 m)

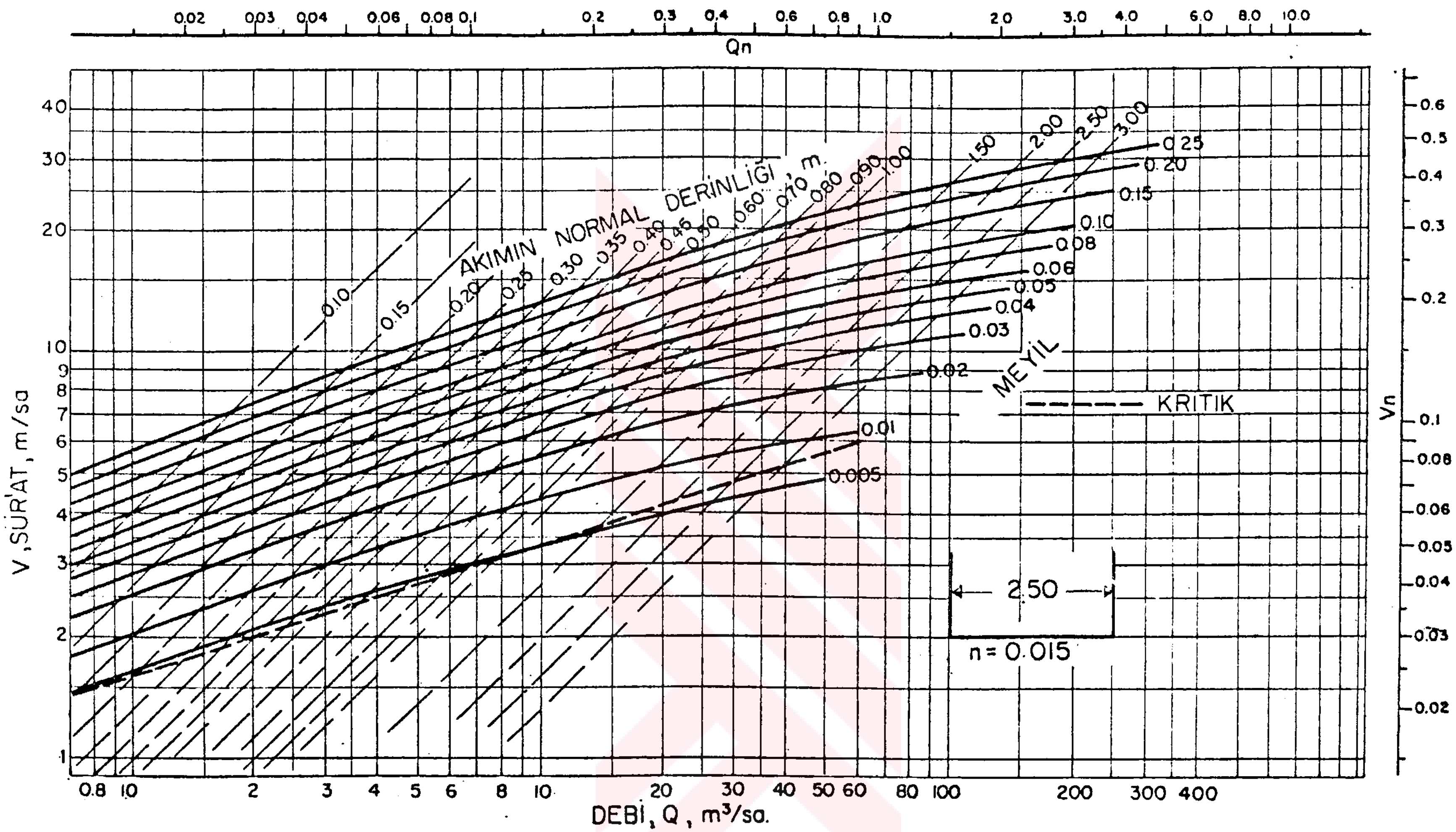


Figure A.16 Discharge in box culverts (for b = 2.50 m)

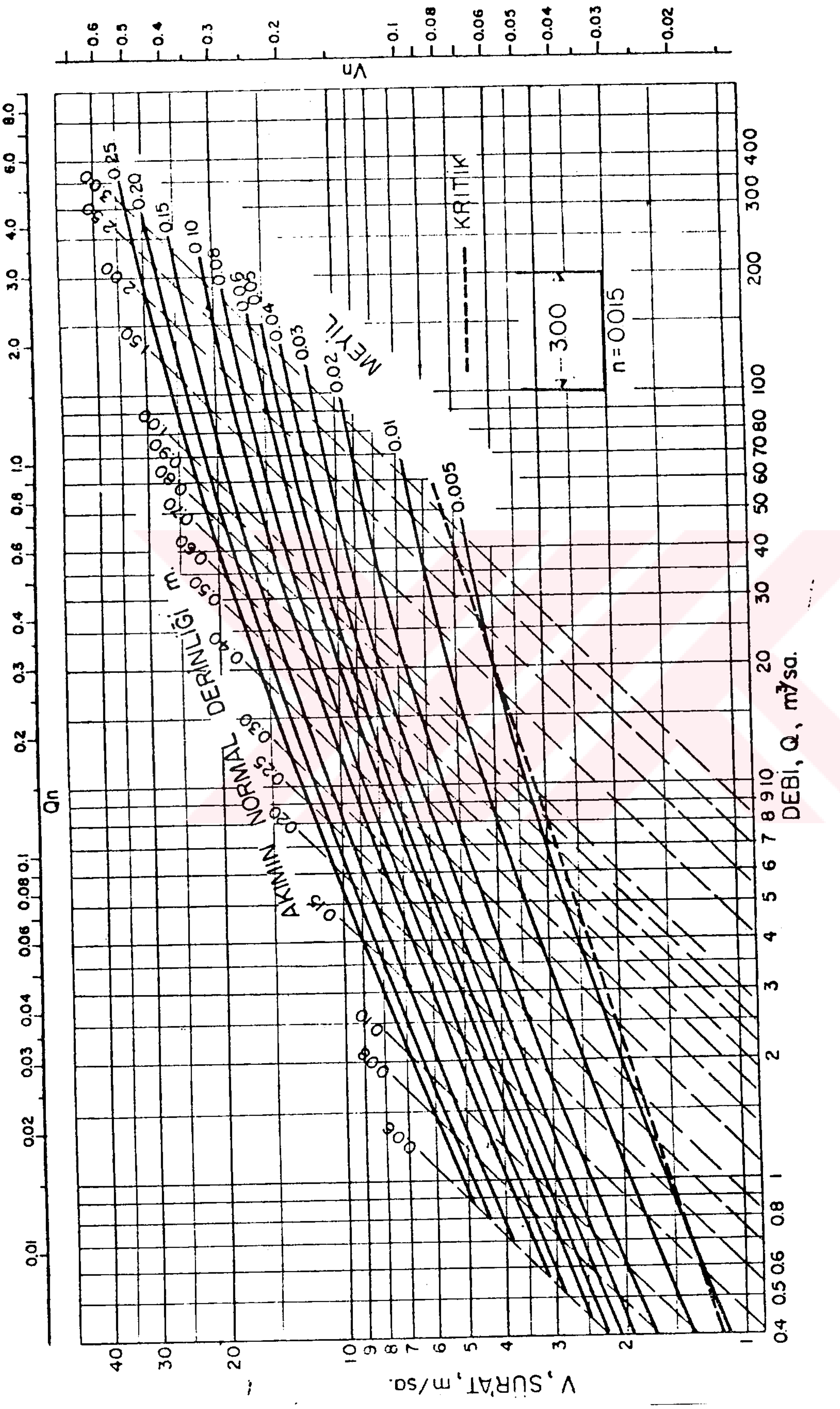


Figure A.17 Discharge in box culverts (for b = 3.00 m)

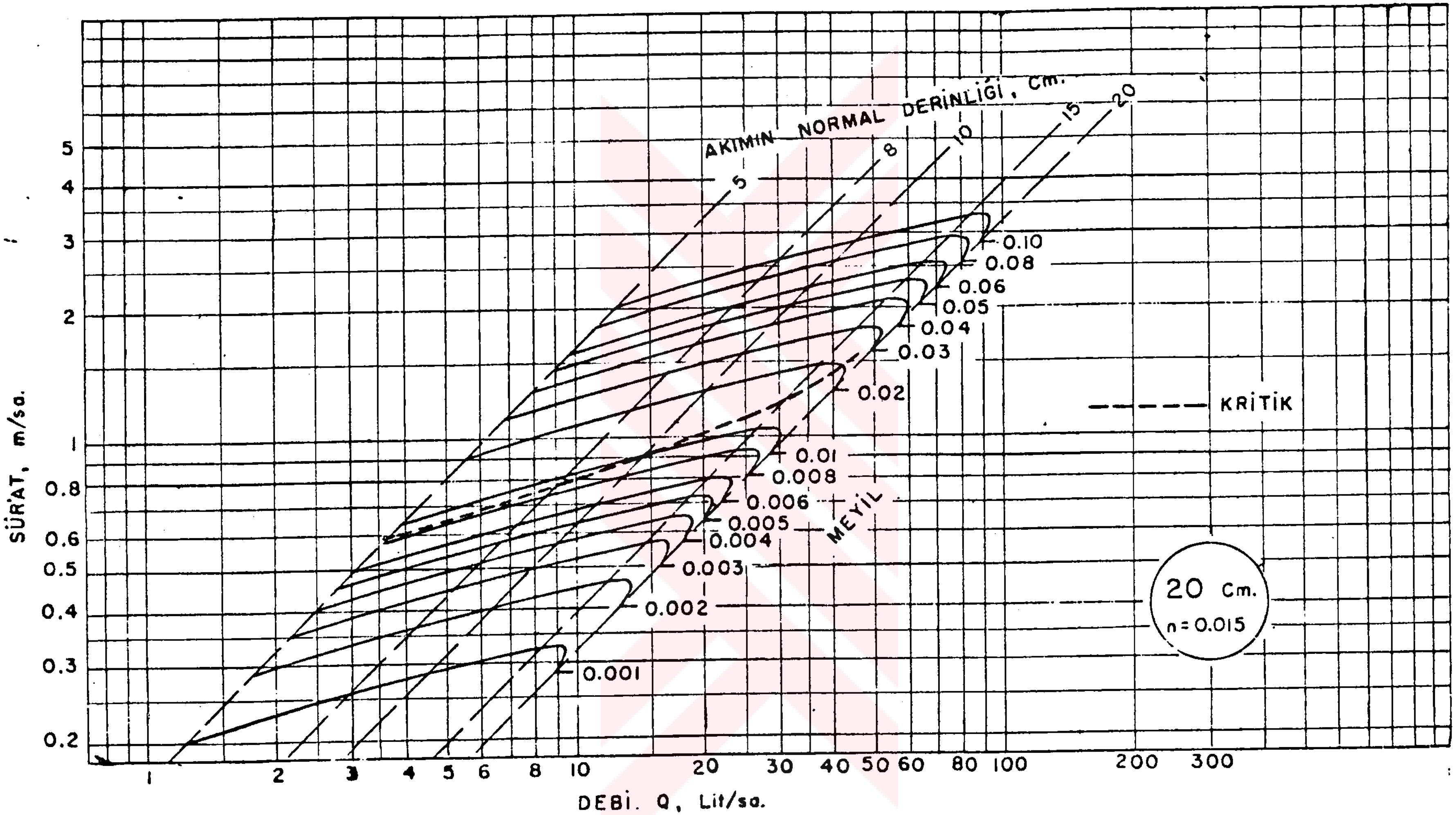


Figure A.18 Discharge in concrete pipe culverts (for D = 20 cm)

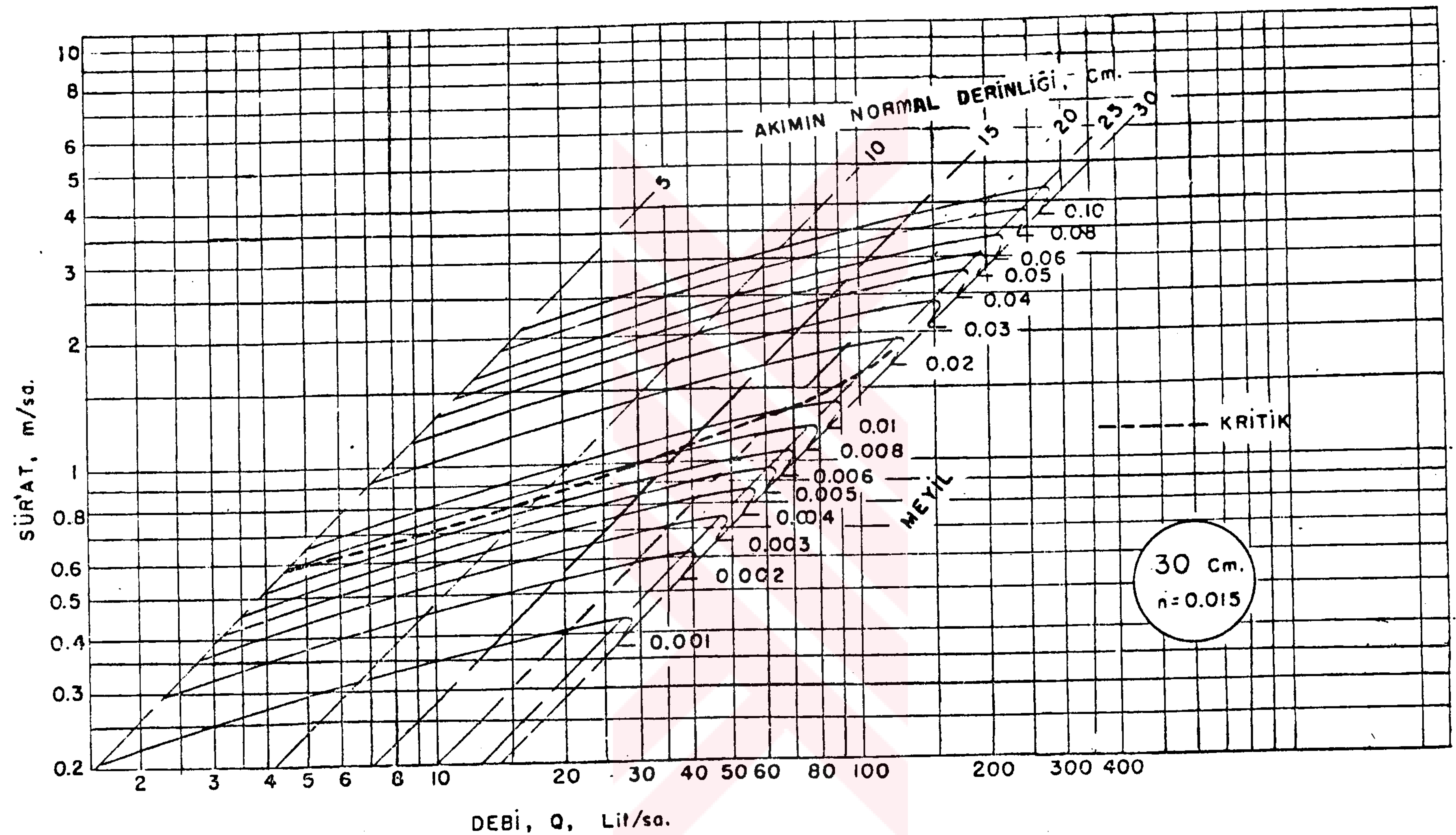


Figure A.19 Discharge in concrete pipe culverts (for D = 30 cm)

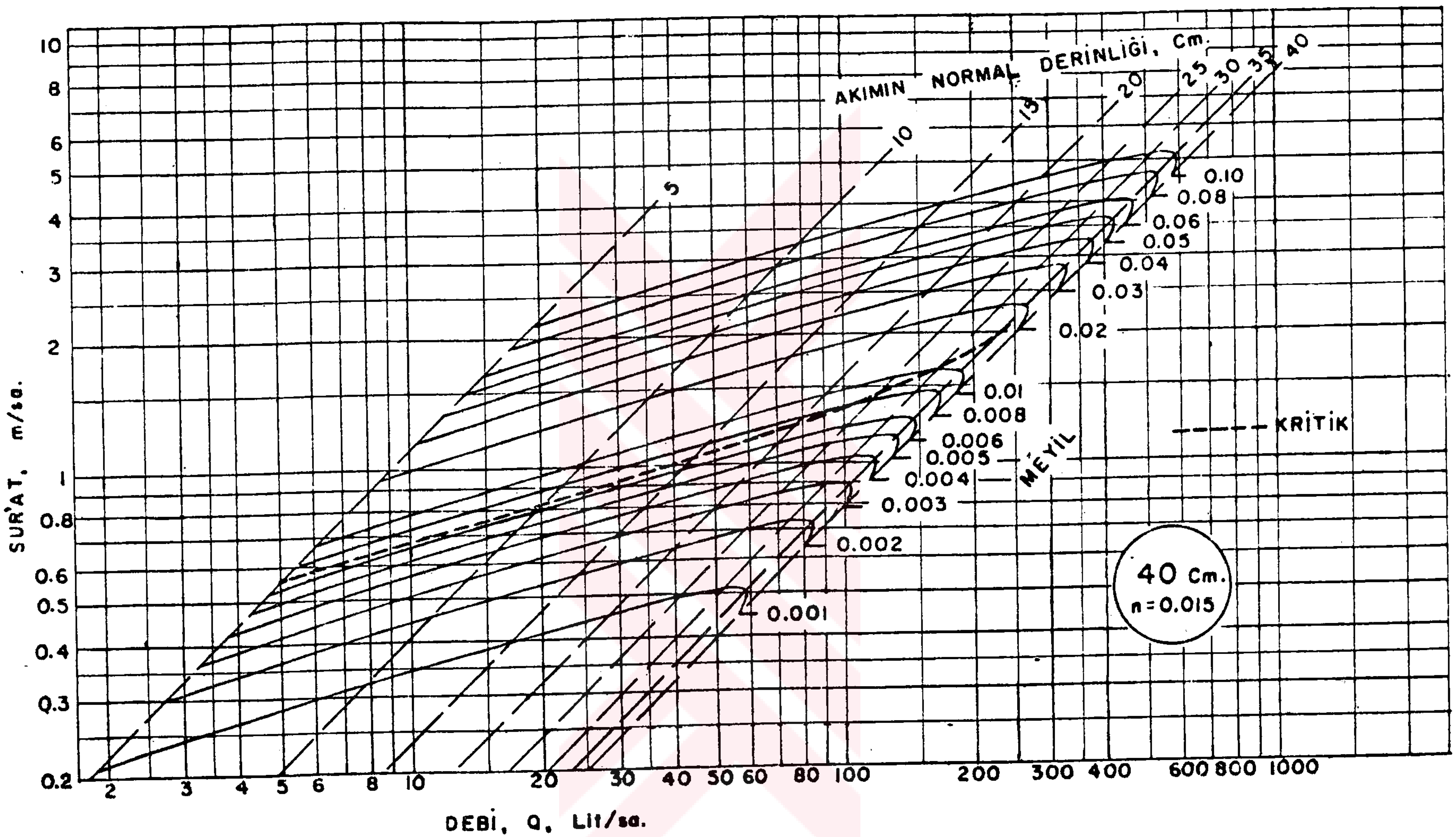


Figure A.20 Discharge in concrete pipe culverts (for D = 40 cm)

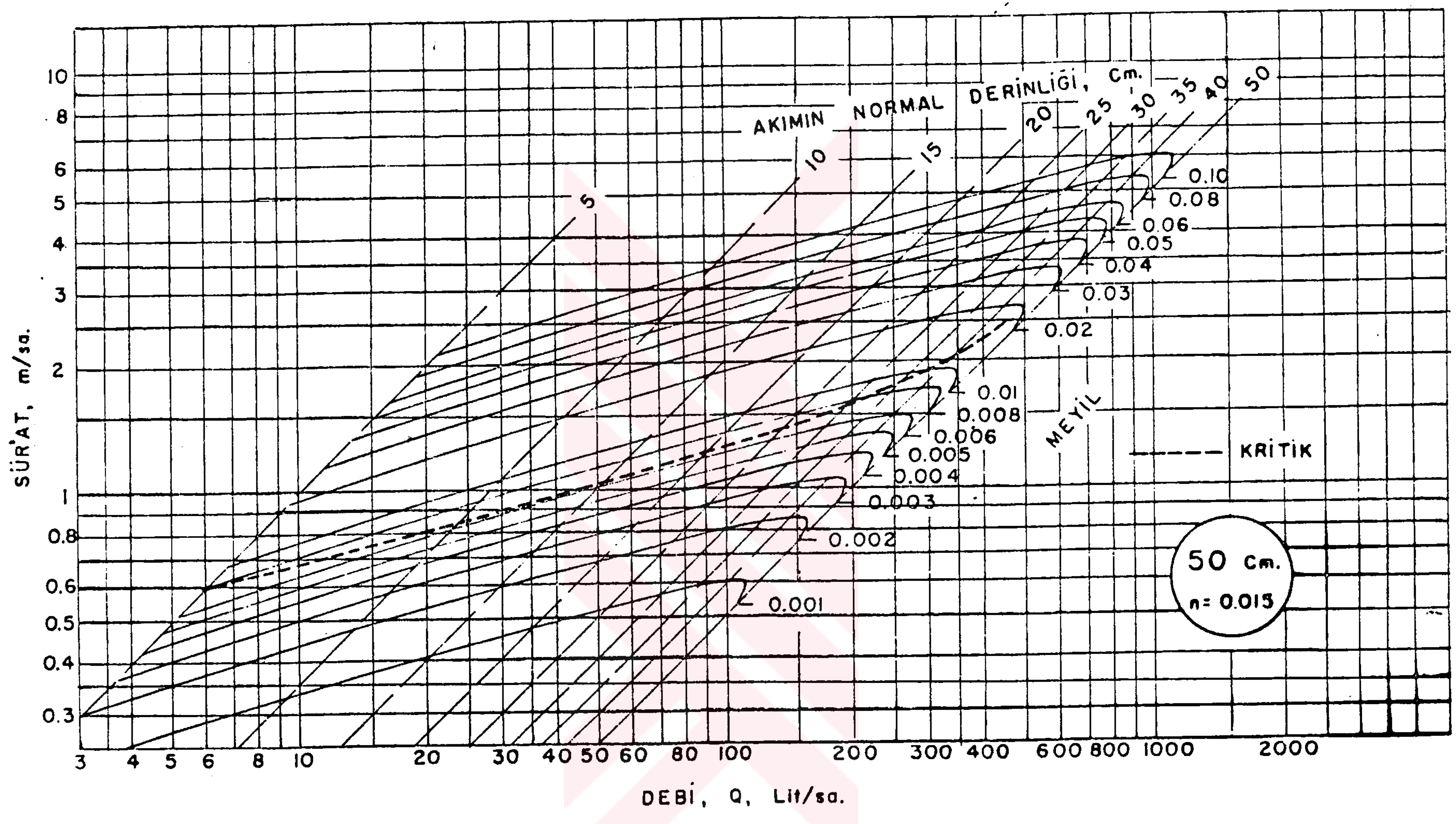


Figure A.21 Discharge in concrete pipe culverts (for D = 50 cm)

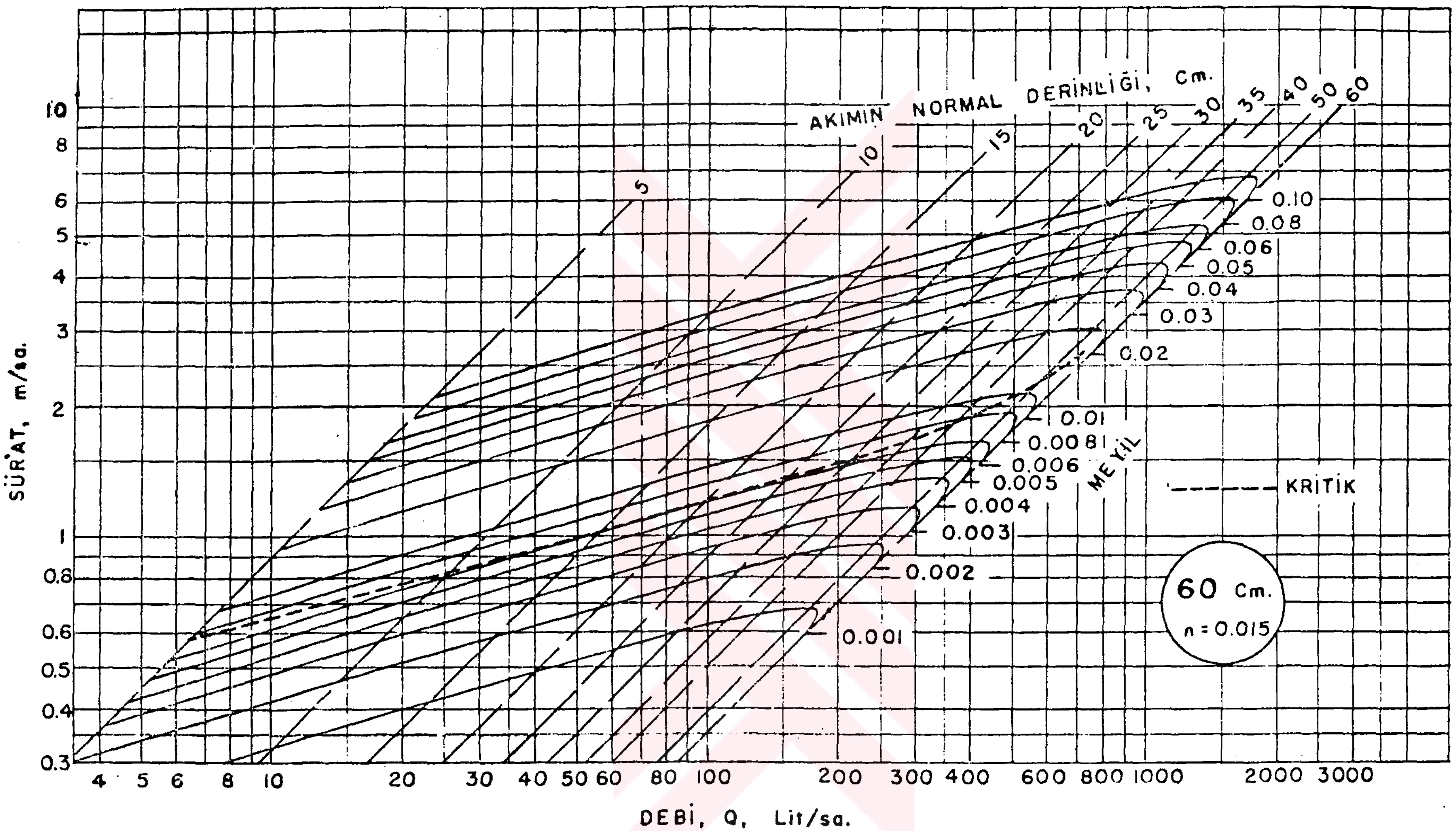


Figure A.22 Discharge in concrete pipe culverts (for D = 60 cm)

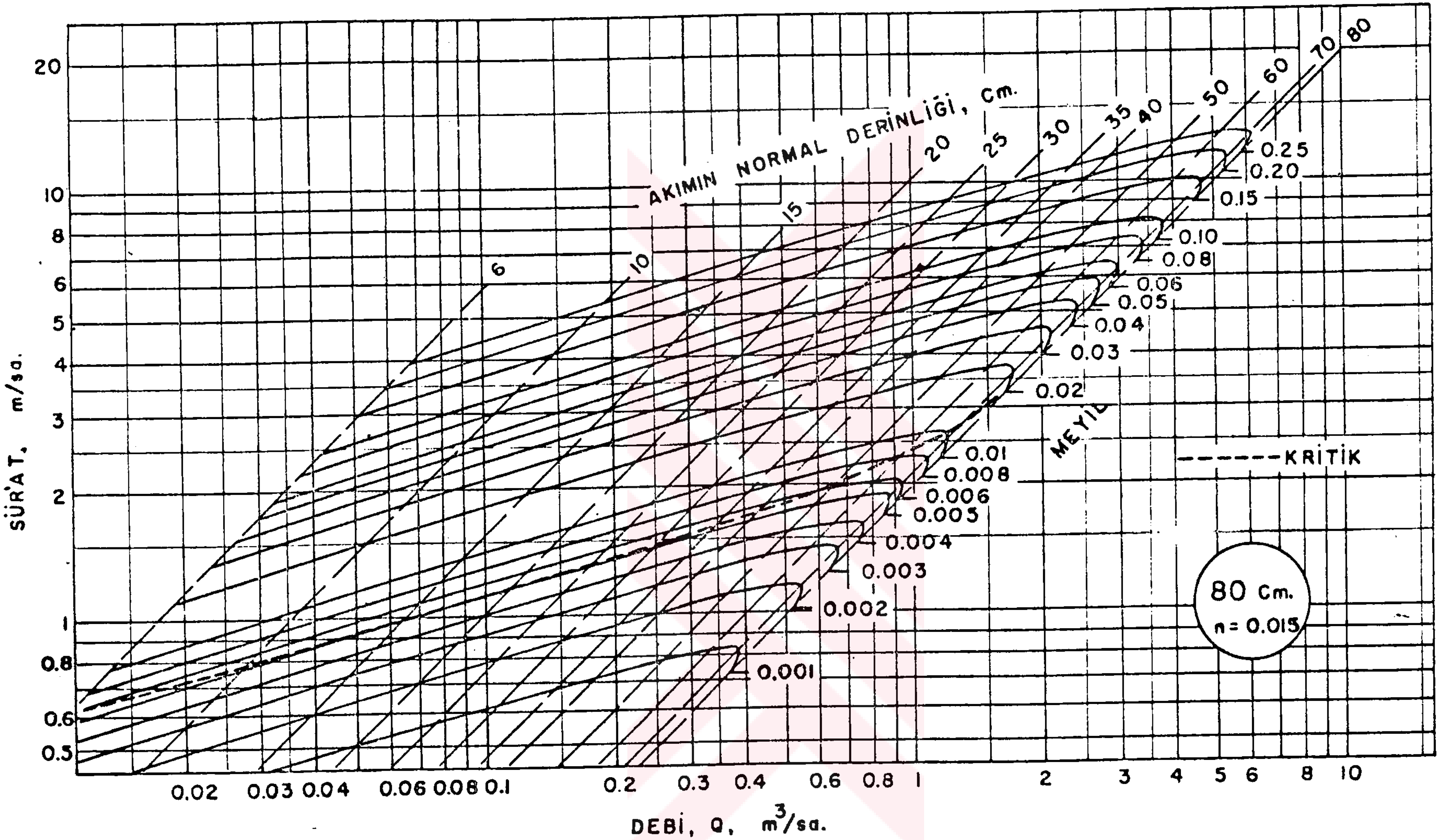
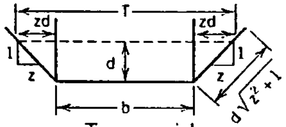
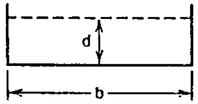
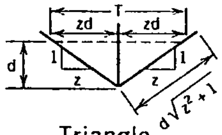
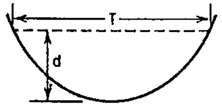
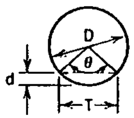
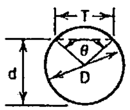


Figure A.23 Discharge in concrete pipe culverts (for D = 80 cm)

APPENDIX B

Table B.1 Hydraulic elements of various cross sections (Mannering&Kilareski, 1998)

Cross Section	Area	Wetted Perimeter	Hydraulic Radius
 Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\dfrac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$
 Rectangle	bd	$b + 2d$	$\dfrac{bd}{b + 2d}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$
 Parabola	$\frac{2}{3}dT$	$T + \dfrac{8d^2}{3T}$	$\dfrac{2dT^2}{3T^2 + 8d^2}$
 Circle < 1/2 full 2	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$	$\dfrac{\pi D\theta}{360}$	$\dfrac{45D}{\pi\theta}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$
 Circle > 1/2 full 3	$\dfrac{D^2}{8}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\dfrac{45D}{\pi(360-\theta)}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$

1. Satisfactory approximation for the interval $0 < \dfrac{d}{T} \leqq 0.25$

   When $\dfrac{d}{T} > 0.25$, use $p = \dfrac{1}{2}\sqrt{16d^2 + T^2} + \dfrac{T^2}{8d}\sinh^{-1}\dfrac{4d}{T}$

2. $\theta = 4\sin^{-1}\sqrt{d/D}$
3. $\theta = 4\cos^{-1}\sqrt{d/D}$

Insert θ in degrees in equations

APPENDIX C

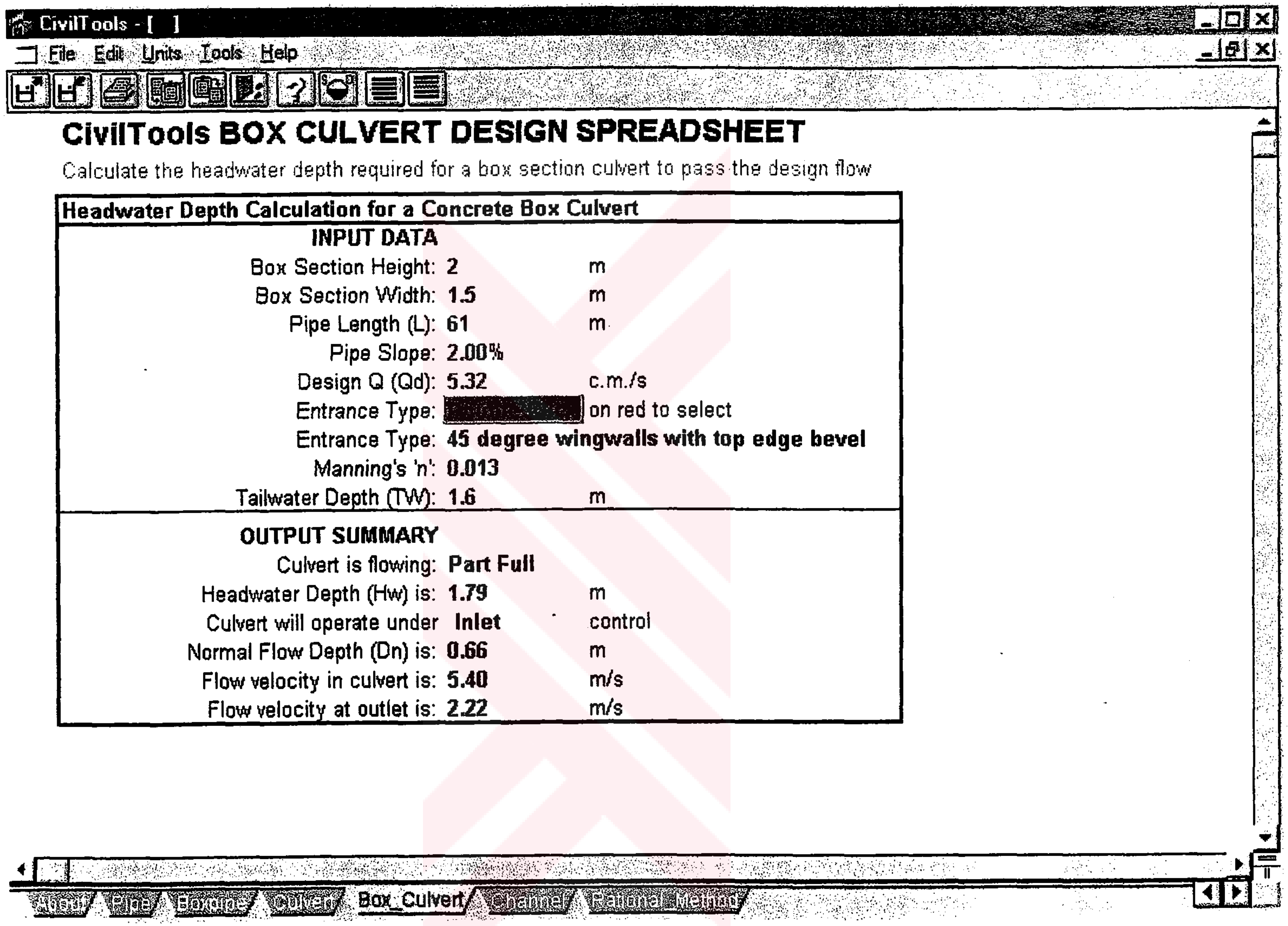


Figure C.1 **CivilTools solution of illustrative example 1 (for $Q_d = 5.32\ m^3/sn$)**

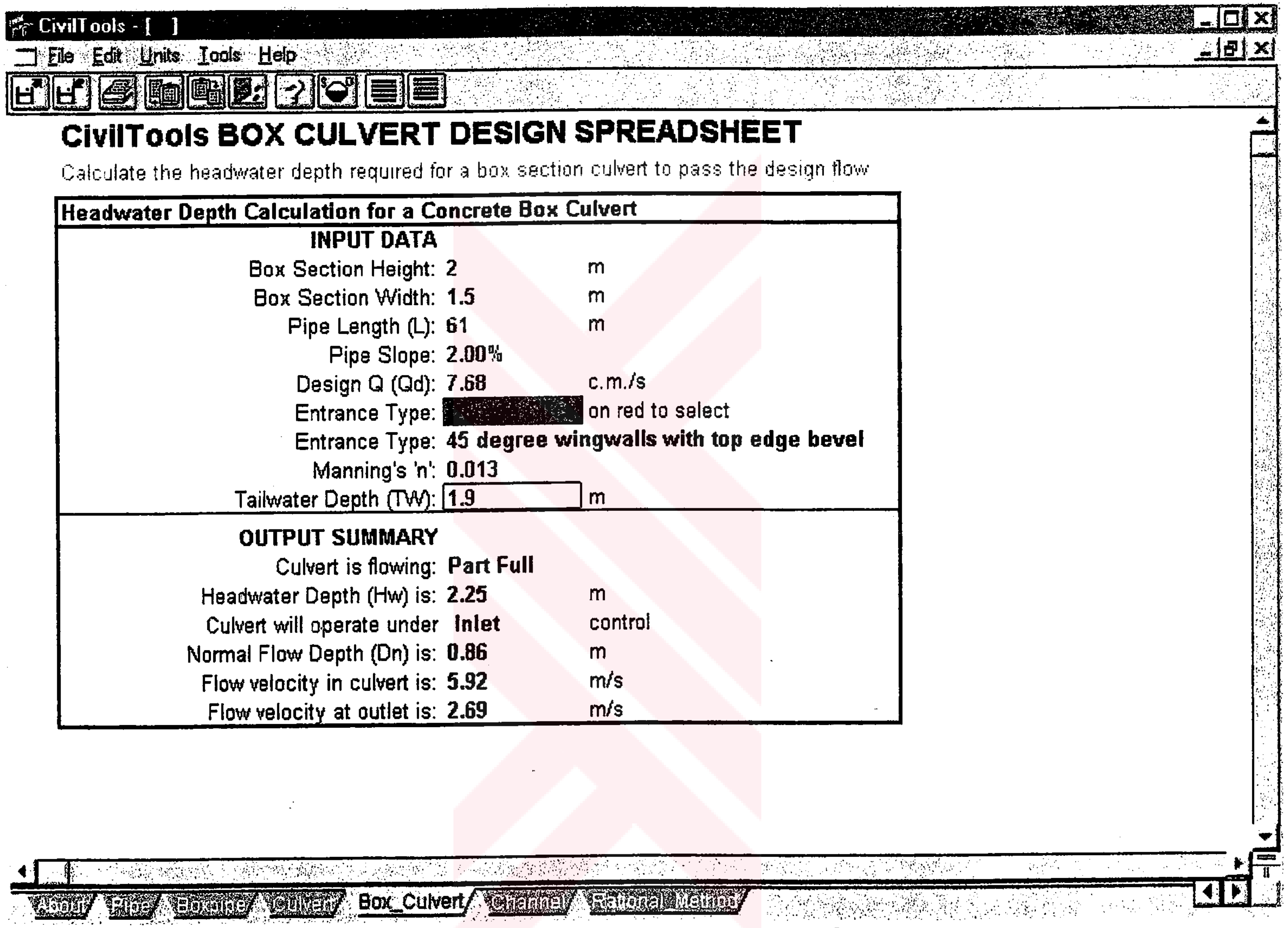


Figure C.2 CivilTools solution of illustrative example 1 (for $Q_d = 7.68\ m^3/sn$)

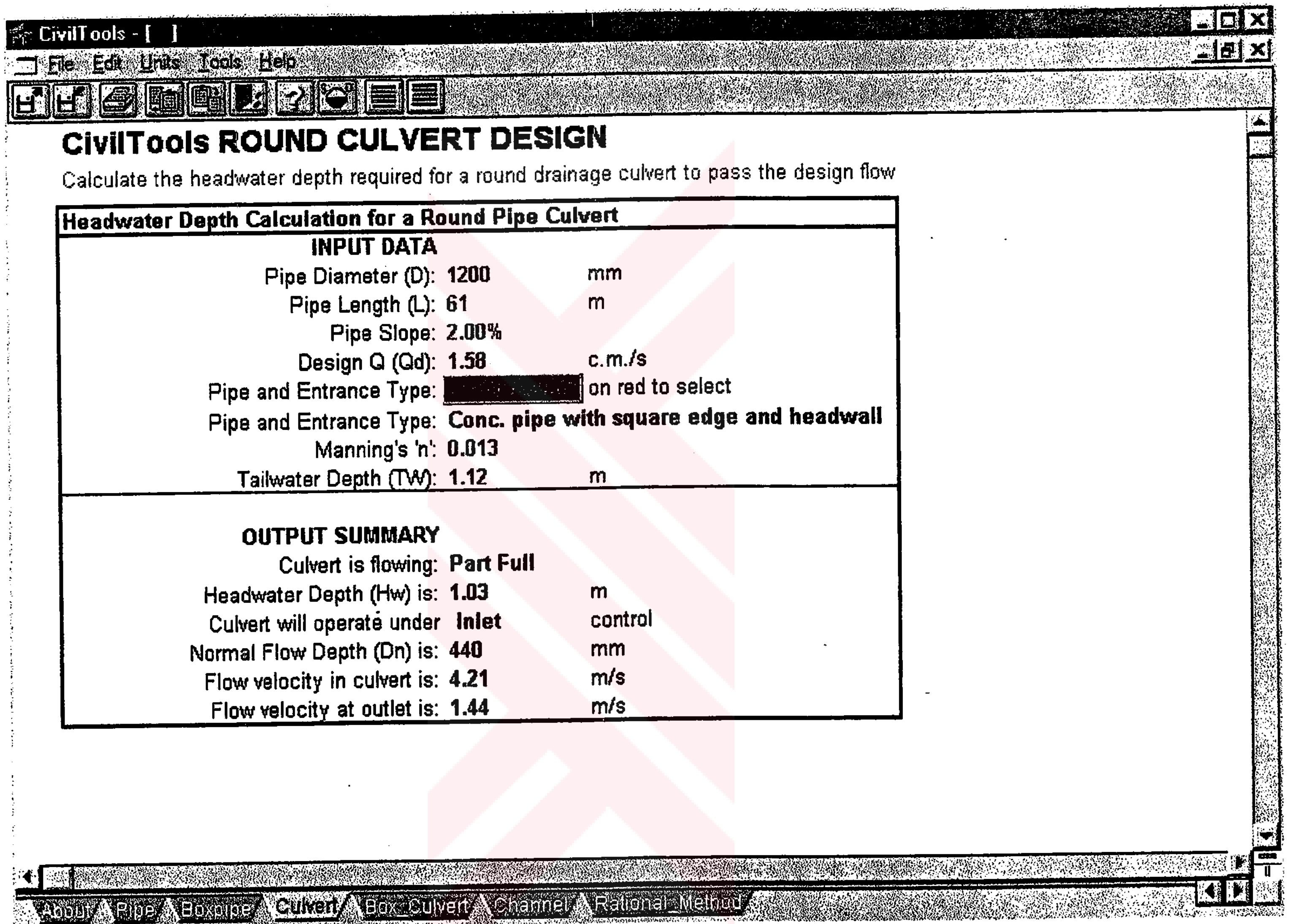


Figure C.3 CivilTools solution of illustrative example 2 (for $Q_d = 1.58$ m^3/sn)

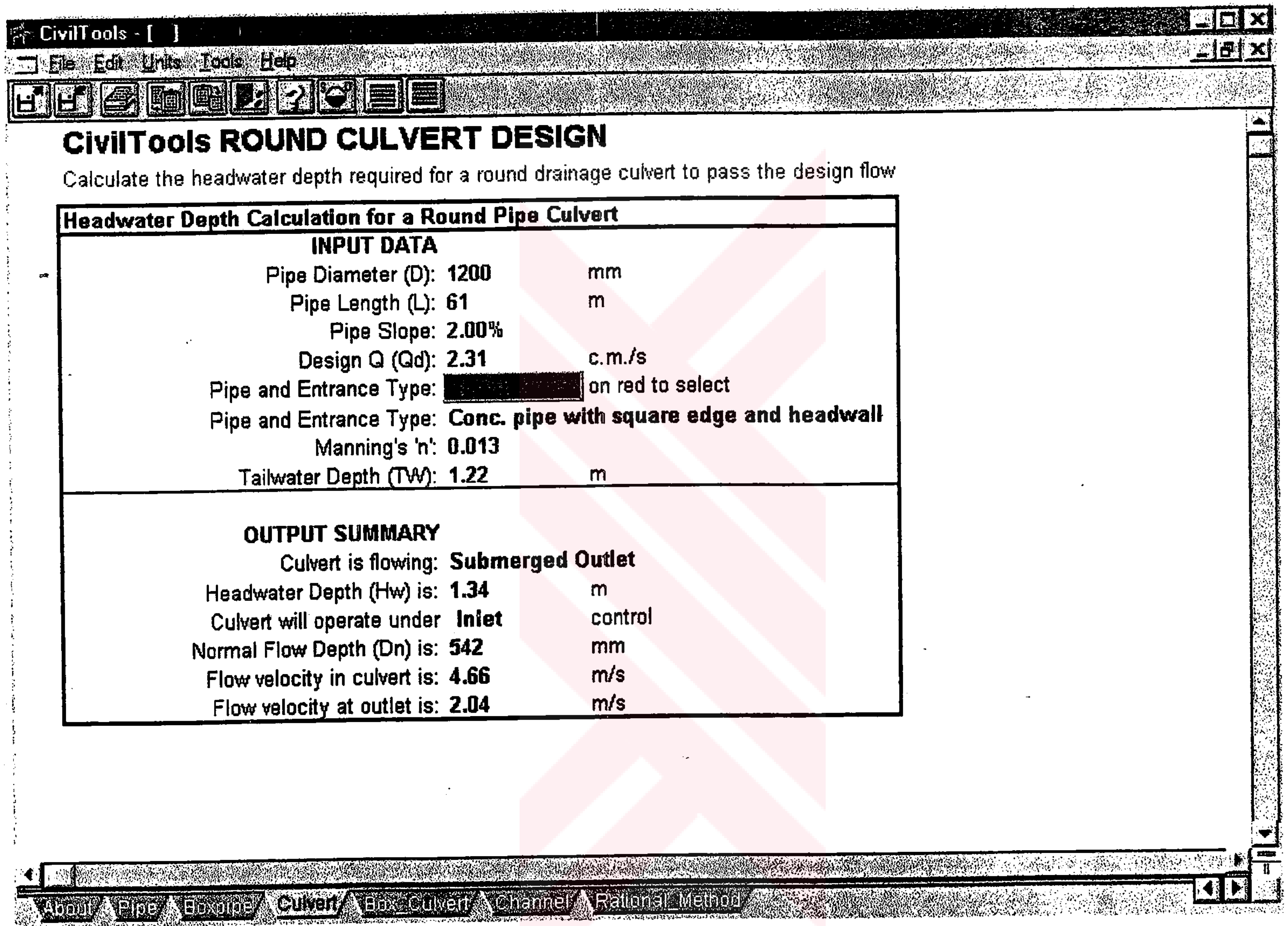


Figure C.4 CivilTools solution of illustrative example 2 (for $Q_d = 2.31\ m^3/sn$)

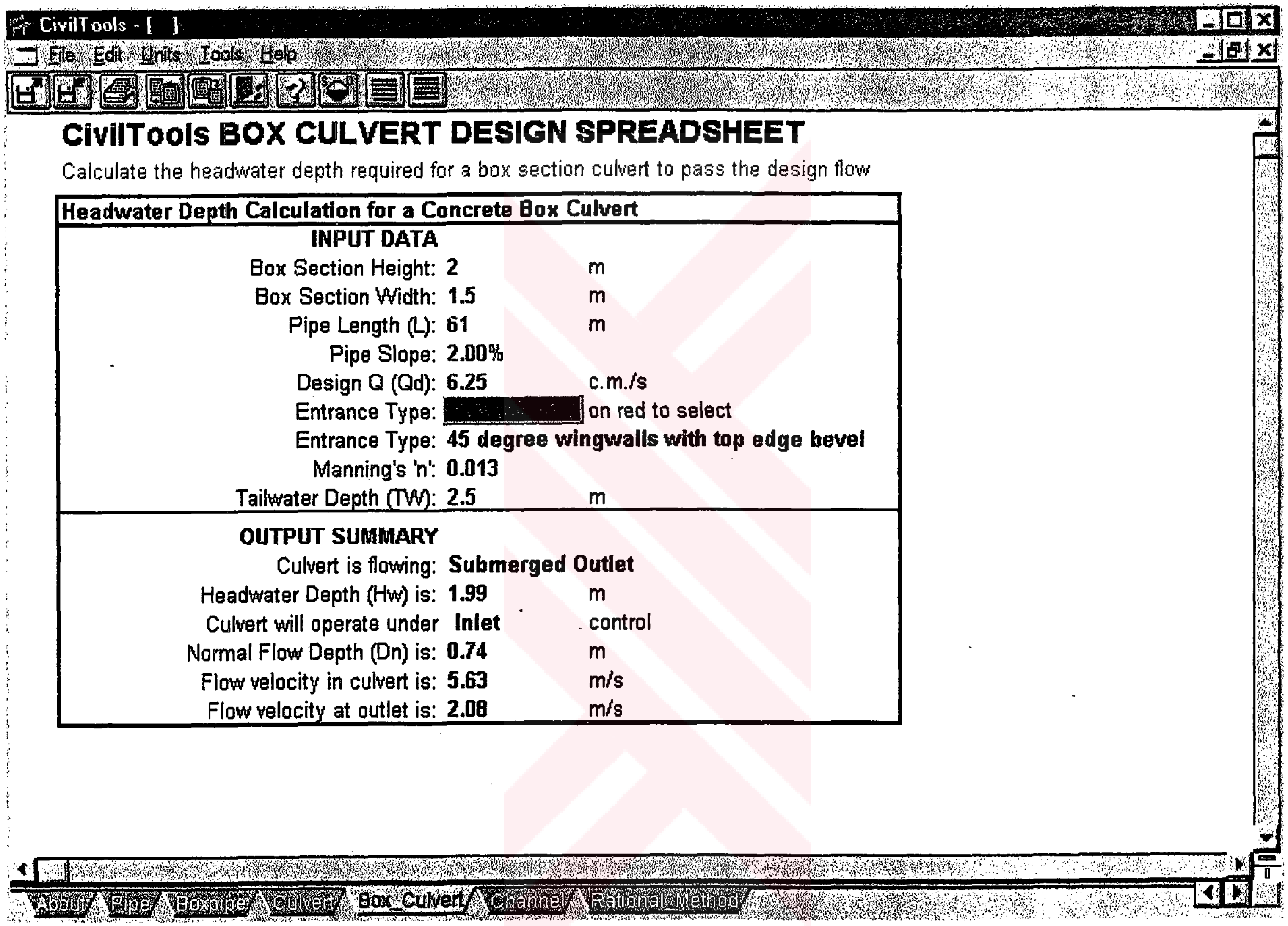


Figure C.5 CivilTools solution of illustrative example 3 (for $Q_d = 12.5\ m^3/sn$)

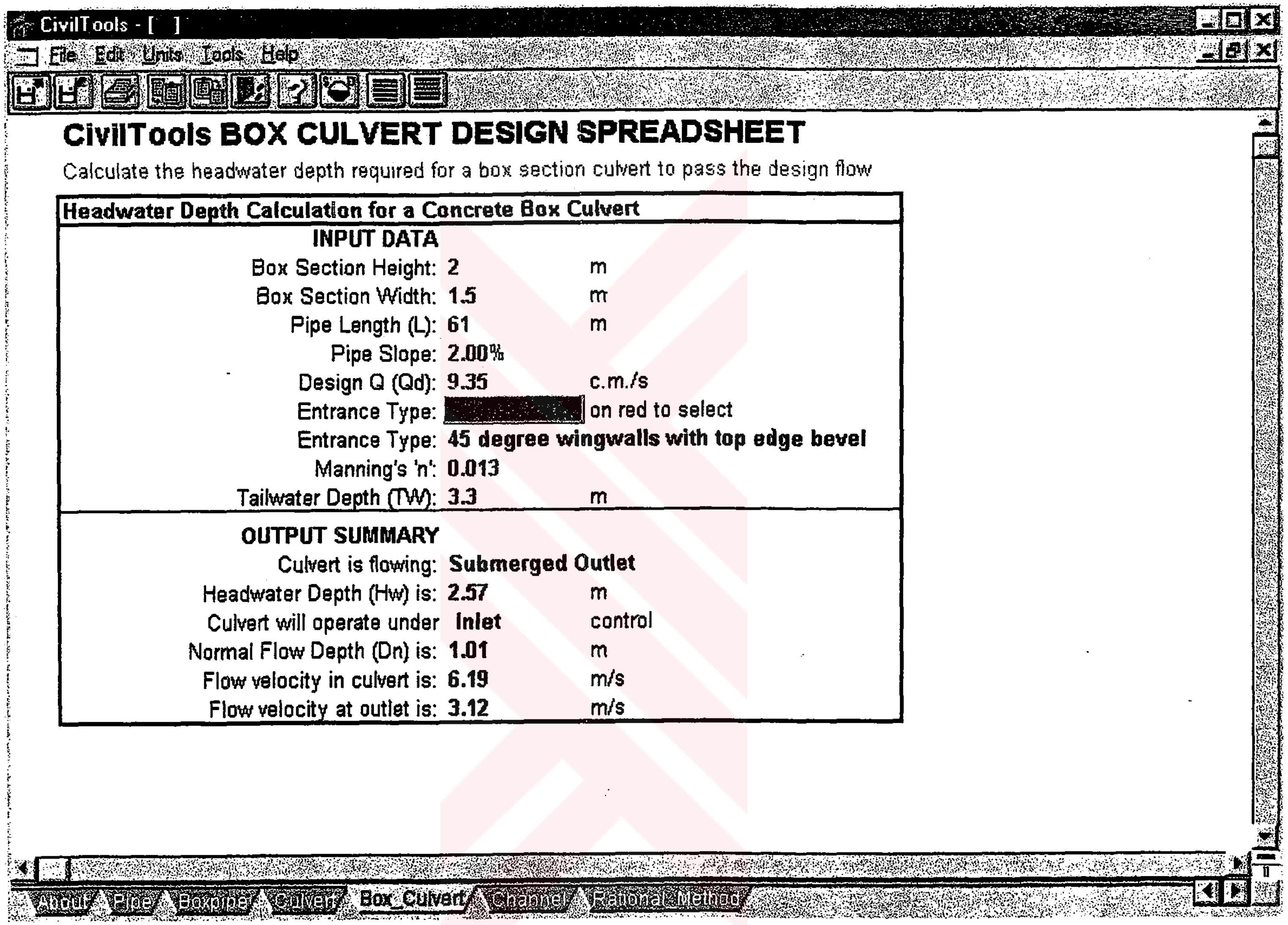


Figure C.6 CivilTools solution of illustrative example 3 (for $Q_d = 18.7\ m^3/sn$)

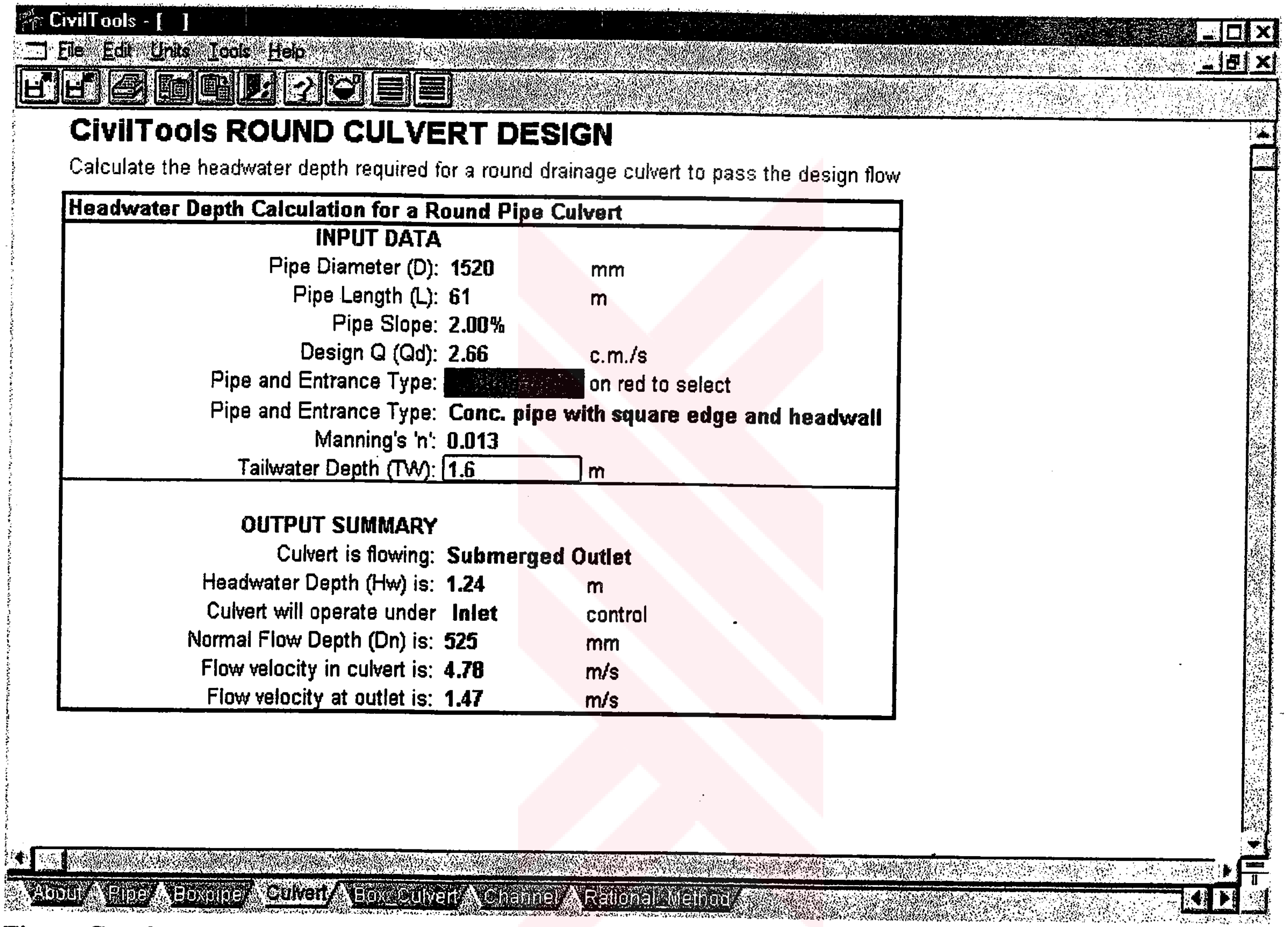


Figure C.7 CivilTools solution of illustrative example 4 (for $Q_d = 5.32\ m^3/sn$)

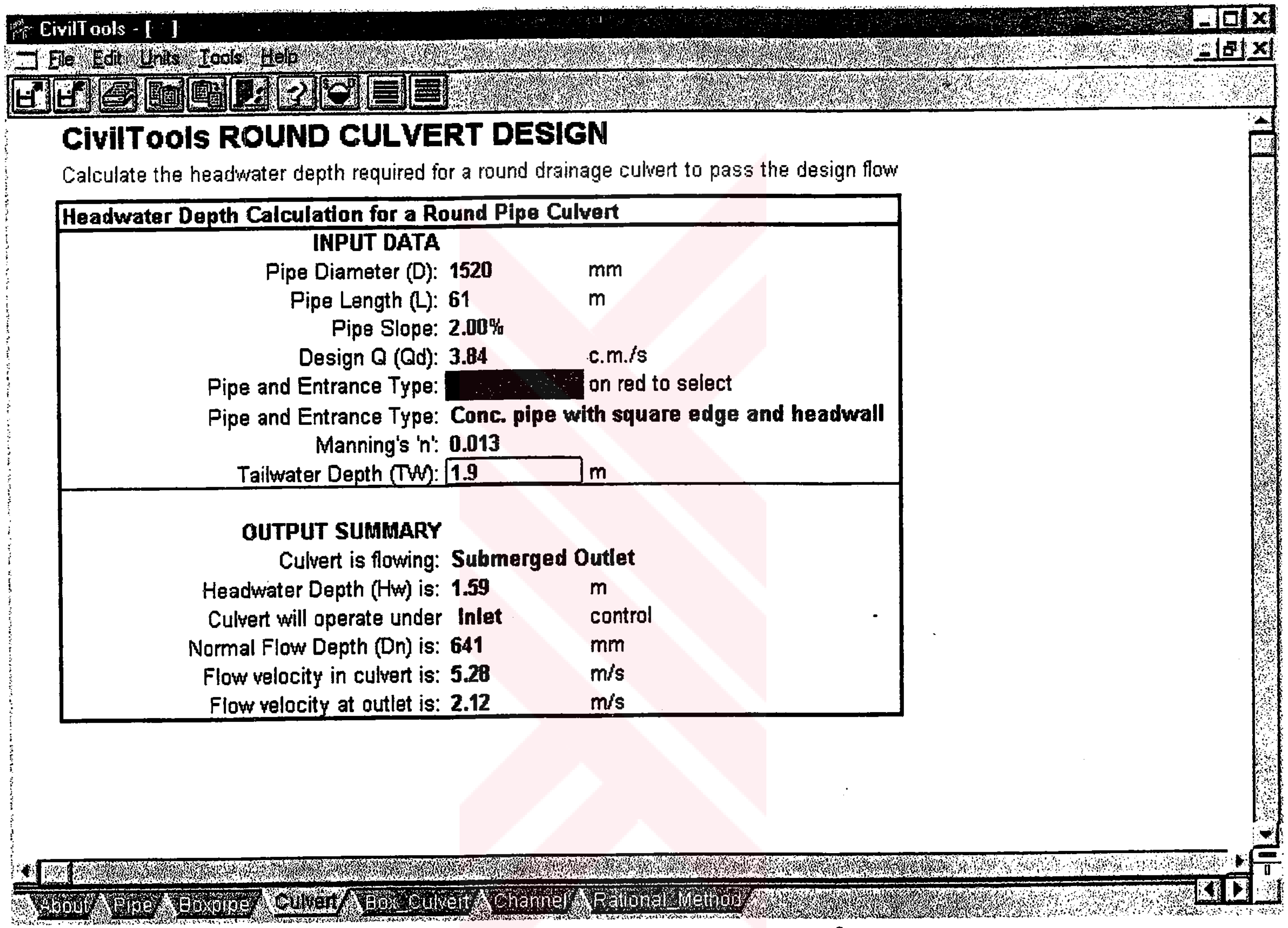


Figure C.8 CivilTools solution of illustrative example 4 (for $Q_d = 7.68\ m^3/sn$)

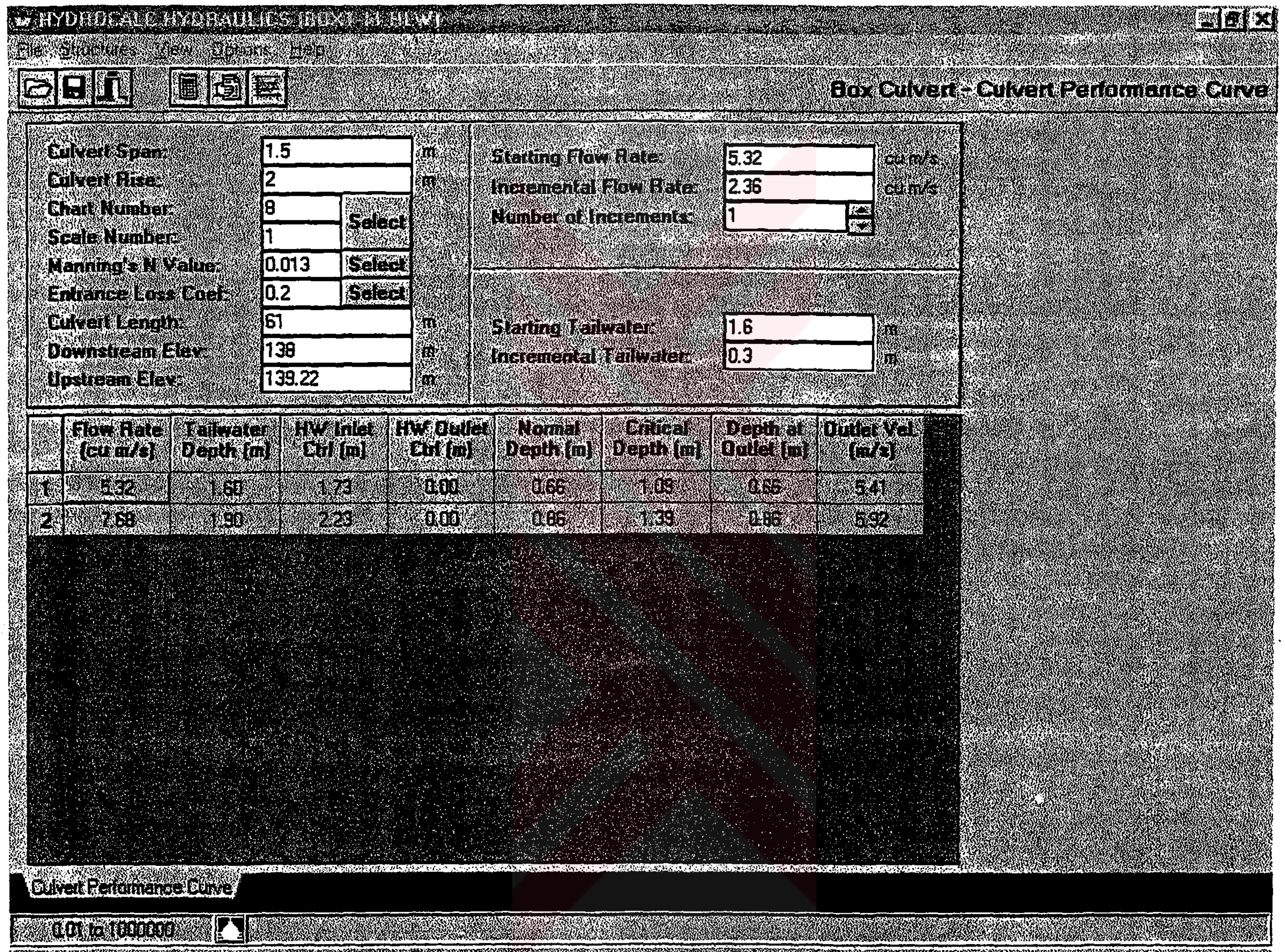


Figure C.9 Dodson HLW solution of illustrative example 1 (for $Q_d = 5.32\ m^3/sn$ and $Q_d = 7.68\ m^3/sn$)

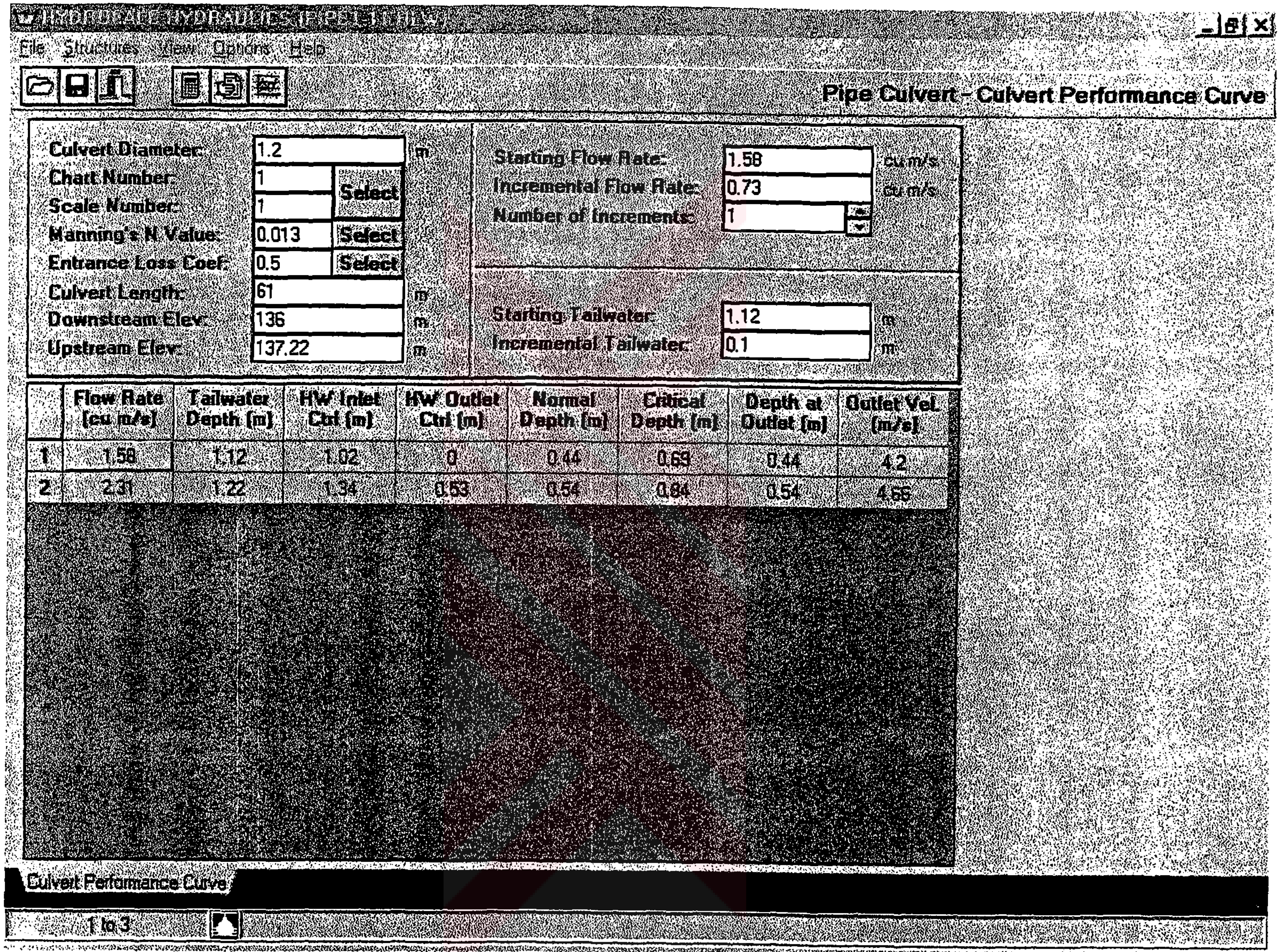


Figure C.10 Dodson HLW solution of illustrative example 2 (for $Q_d = 1.58$ m^3/sn and $Q_d = 2.31$ m^3/sn)

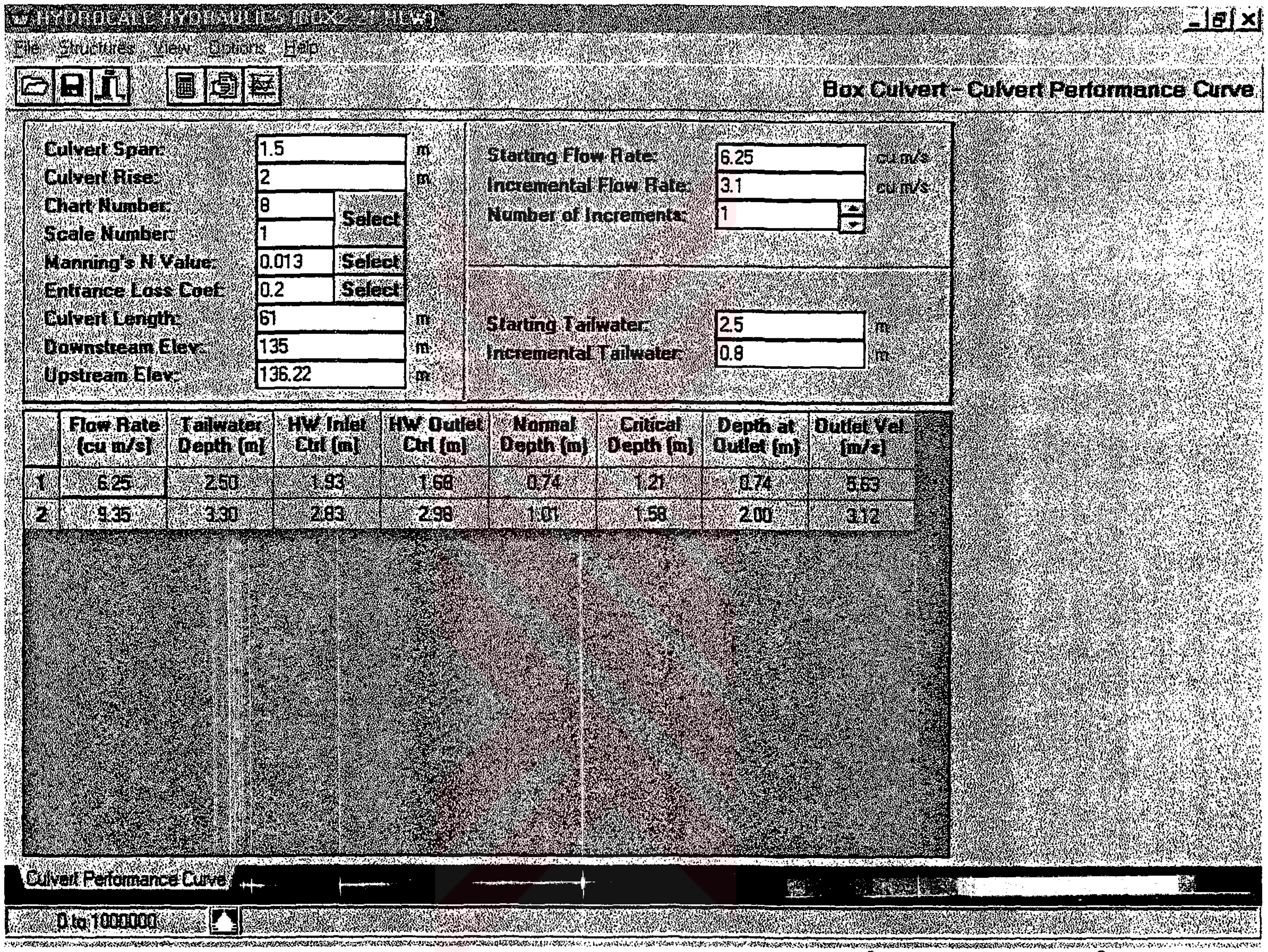


Figure C.11 Dodson HLW solution of illustrative example 3 (for $Q_d = 12.5\ m^3/sn$ and $Q_d = 18.7\ m^3/sn$)

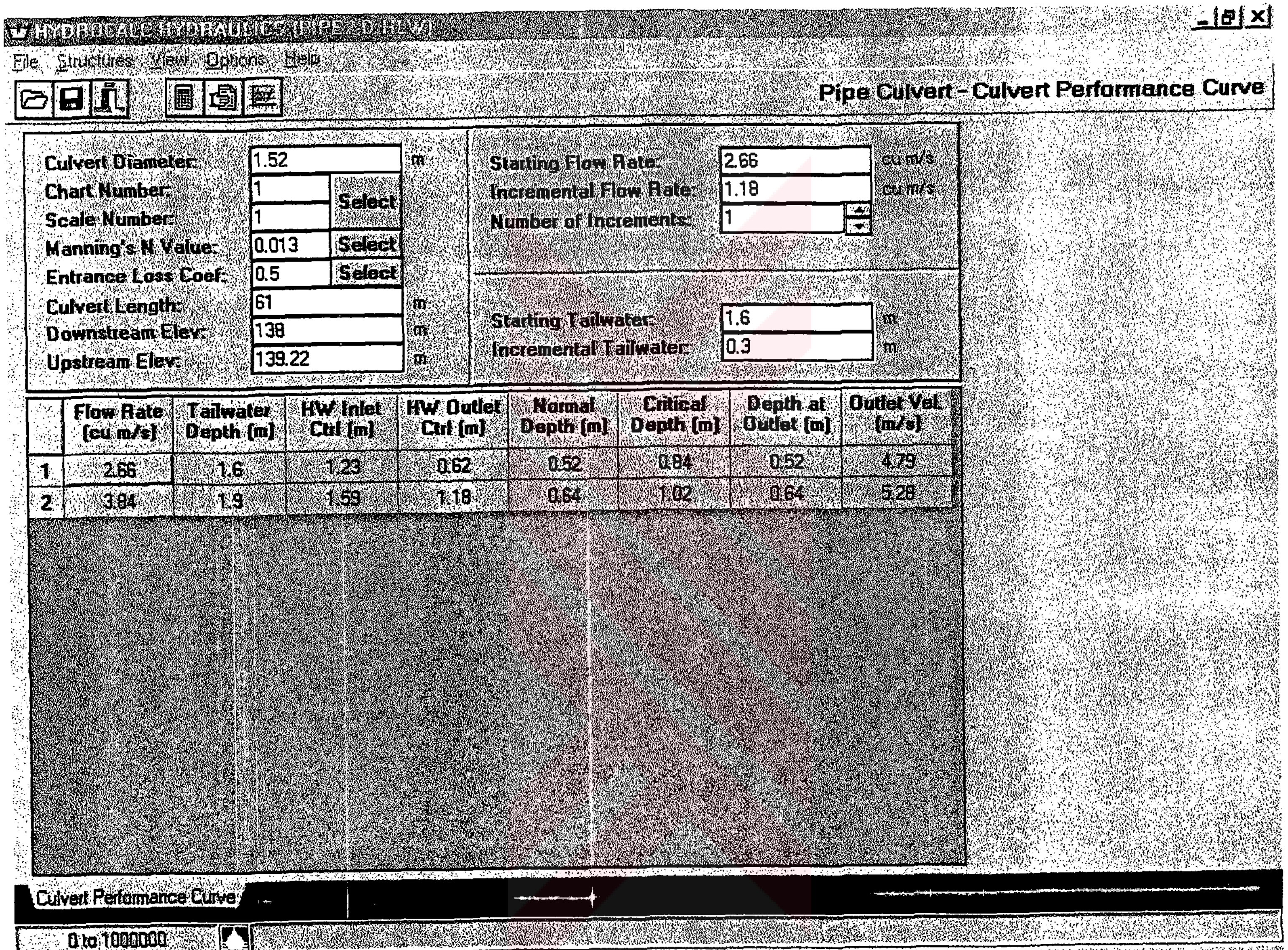


Figure C.12 Dodson HLW solution of illustrative example 4 (for $Q_d = 5.32$ m^3/sn and $Q_d = 7.68$ m^3/sn)

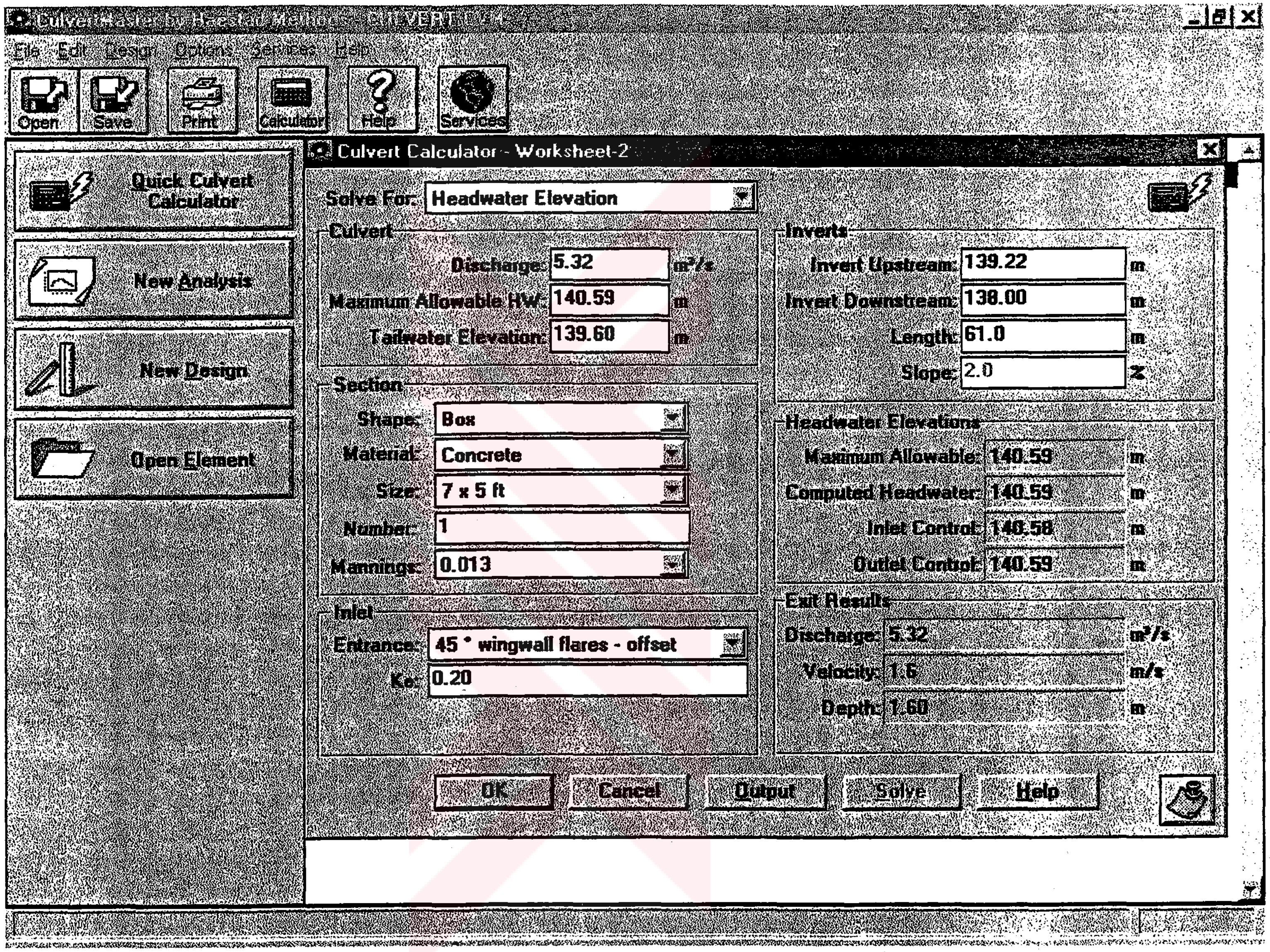


Figure C.13 CulvertMaster solution of illustrative example 1 (for $Q_d = 5.32\ m^3/sn$)

Performance Curves Report
Worksheet-1

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	11.00	1.10 m³/s

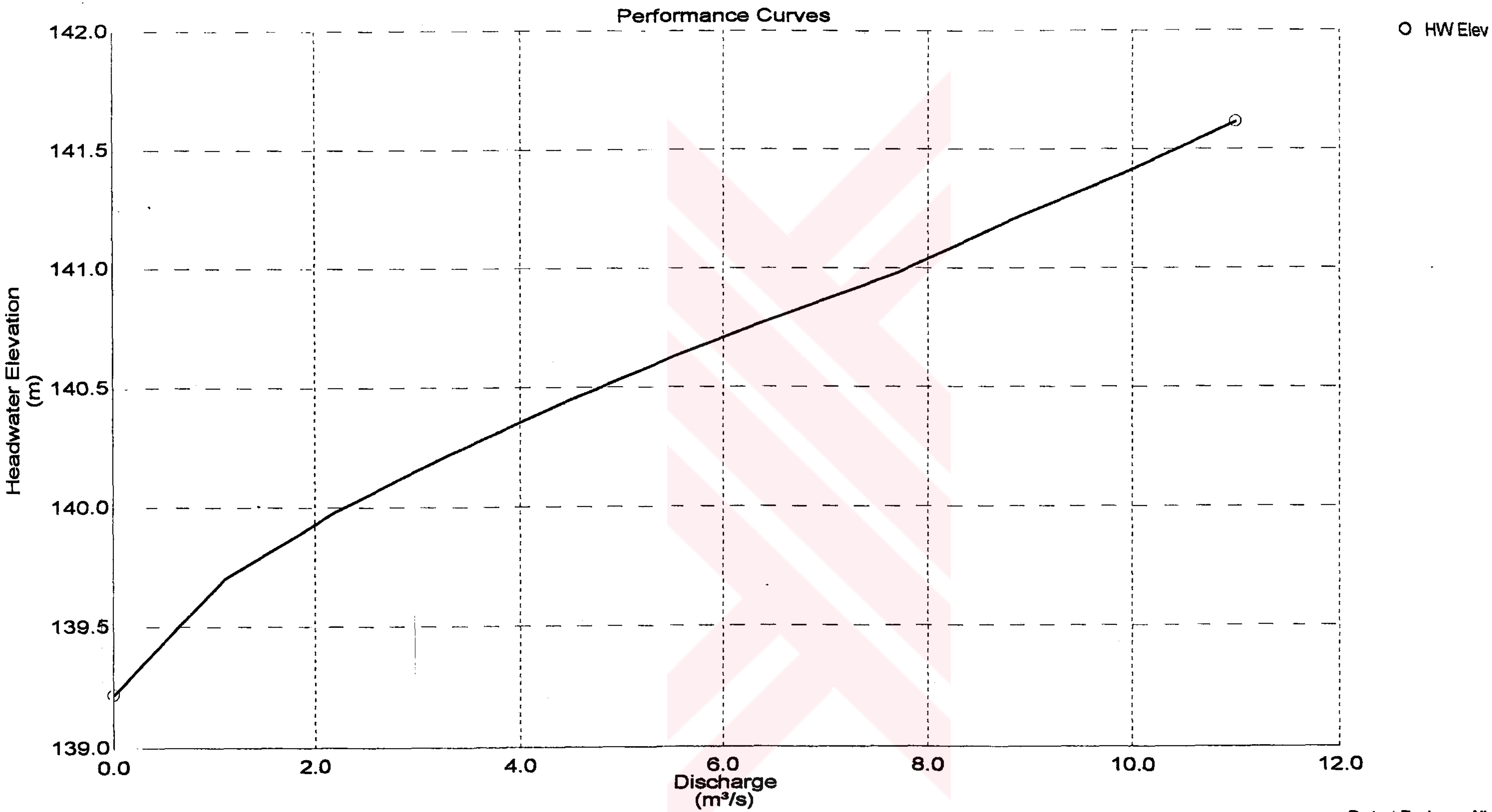


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Figure C.14 CulvertMaster performance curves report for $Q_d = 5.32$ m^3/sec

Culvert Calculator Report
Worksheet-1

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	140.59 m	Headwater Depth/ Height	0.90
Computed Headwater Elevation	140.59 m	Discharge	5.32 m^3/s
Inlet Control HW Elev	140.58 m	Tailwater Elevation	139.60 m
Outlet Control HW Elev	140.59 m	Control Type	Entrance Control

Grades			
Upstream Invert	139.22 m	Downstream Invert	138.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	CompositePressureS1S2	Depth, Downstream	1.60 m
Slope Type	N/A	Normal Depth	0.48 m
Flow Regime	N/A	Critical Depth	0.86 m
Velocity Downstream	1.6 m/s	Critical Slope	0.4 %

Section			
Section Shape	Box	Mannings Coefficient	0.013
Section Material	Concrete	Span	7.00 ft
Section Size	7 x 5 ft	Rise	5.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	140.59 m	Upstream Velocity Head	0.43 m
Ke	0.20	Entrance Loss	0.09 m

Inlet Control Properties			
Inlet Control HW Elev	140.58 m	Flow Control	N/A
Inlet Type	45 ° wingwall flares - offset	Area Full	35.0 ft^2
K	0.49700	HDS 5 Chart	13
M	0.66700	HDS 5 Scale	1
C	0.03020	Equation Form	2
Y	0.83500		

Figure C.15 CulvertMaster report for $Q_d = 5.32\ m^3/sec$

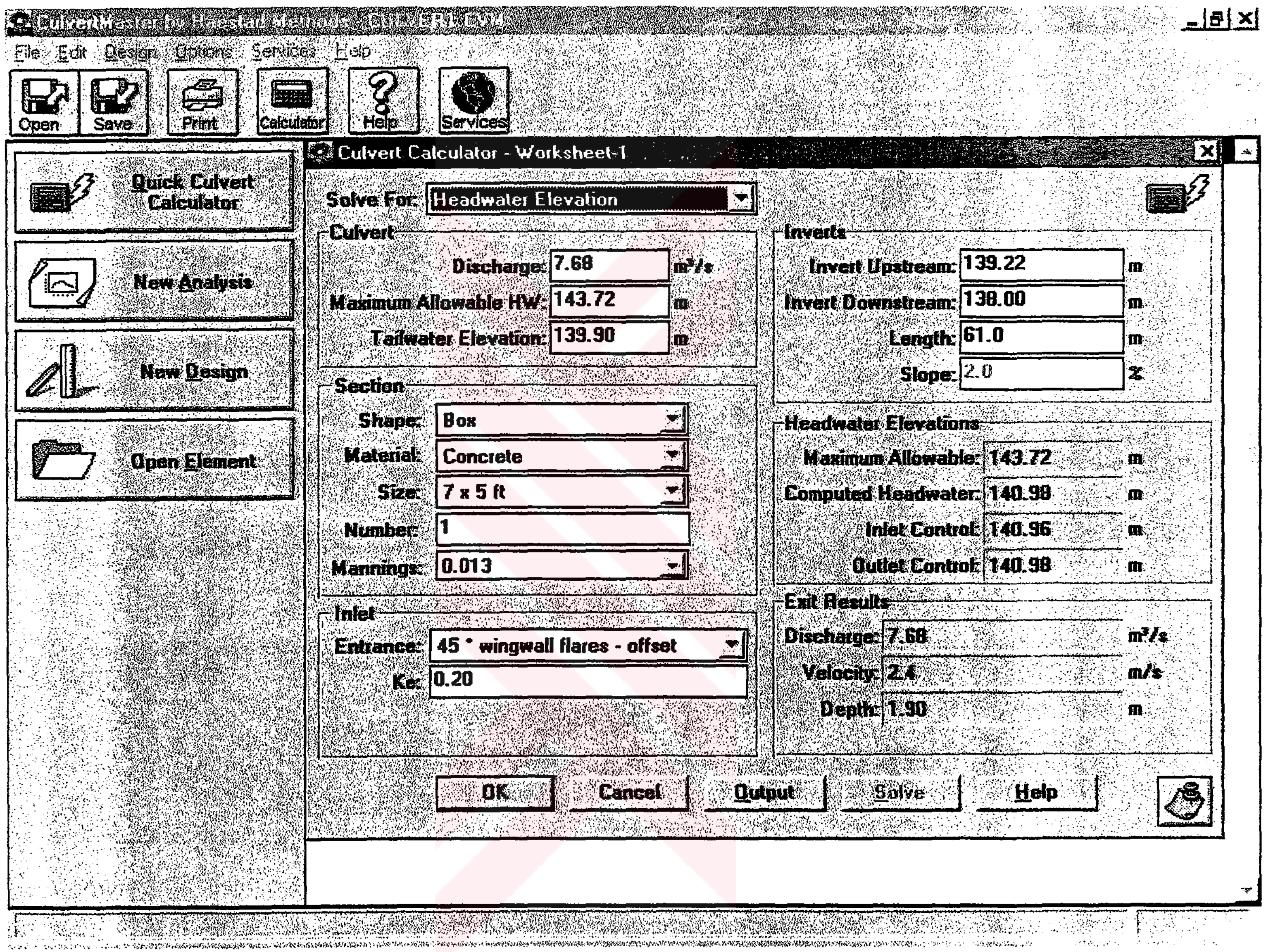

Figure C.16 CulvertMaster solution of illustrative example 1 (for $Q_d = 7.68$ m^3/sn)

Performance Curves Report
Worksheet-2

Minimum	Maximum	Increment
0.00	15.00	1.50 m³/s

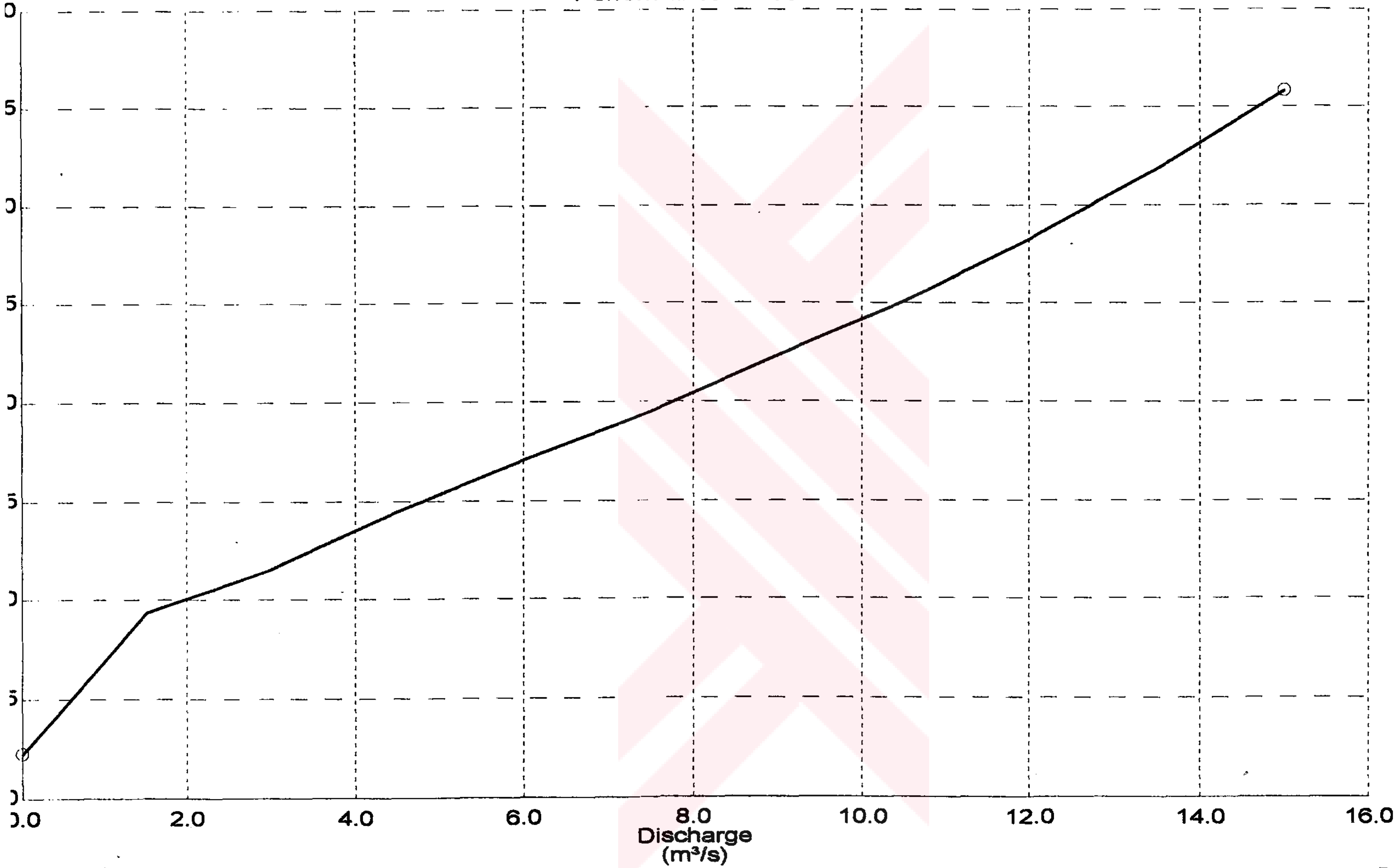

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Figure C.17 CulvertMaster performance curves report for $Q_d = 7.68\ m^3/sec$

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Culvert Calculator Report
Worksheet-2

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	143.72	m	Headwater Depth/ Height	1.15	
Computed Headwater Elevation	140.98	m	Discharge	7.68	m³/s
Inlet Control HW Elev	140.96	m	Tailwater Elevation	139.90	m
Outlet Control HW Elev	140.98	m	Control Type	Entrance Control	

Grades					
Upstream Invert	139.22	m	Downstream Invert	138.00	m
Length	61.0	m	Constructed Slope	2.0	%

Hydraulic Profile					
Profile	CompositePressureS1S2		Depth, Downstream	1.90	m
Slope Type	N/A		Normal Depth	0.62	m
Flow Regime	N/A		Critical Depth	1.10	m
Velocity Downstream	2.4	m/s	Critical Slope	0.4	%

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	7.00	ft
Section Size	7 x 5 ft		Rise	5.00	ft
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev	140.98	m	Upstream Velocity Head	0.55	m
Ke	0.20		Entrance Loss	0.11	m

Inlet Control Properties					
Inlet Control HW Elev	140.96	m	Flow Control	N/A	
Inlet Type	45 ° wingwall flares - offset		Area Full	35.0	ft²
K	0.49700		HDS 5 Chart	13	
M	0.66700		HDS 5 Scale	1	
C	0.03020		Equation Form	2	
Y	0.83500				

Figure C.18 CulvertMaster report for $Q_d = 7.68$ m^3/sec

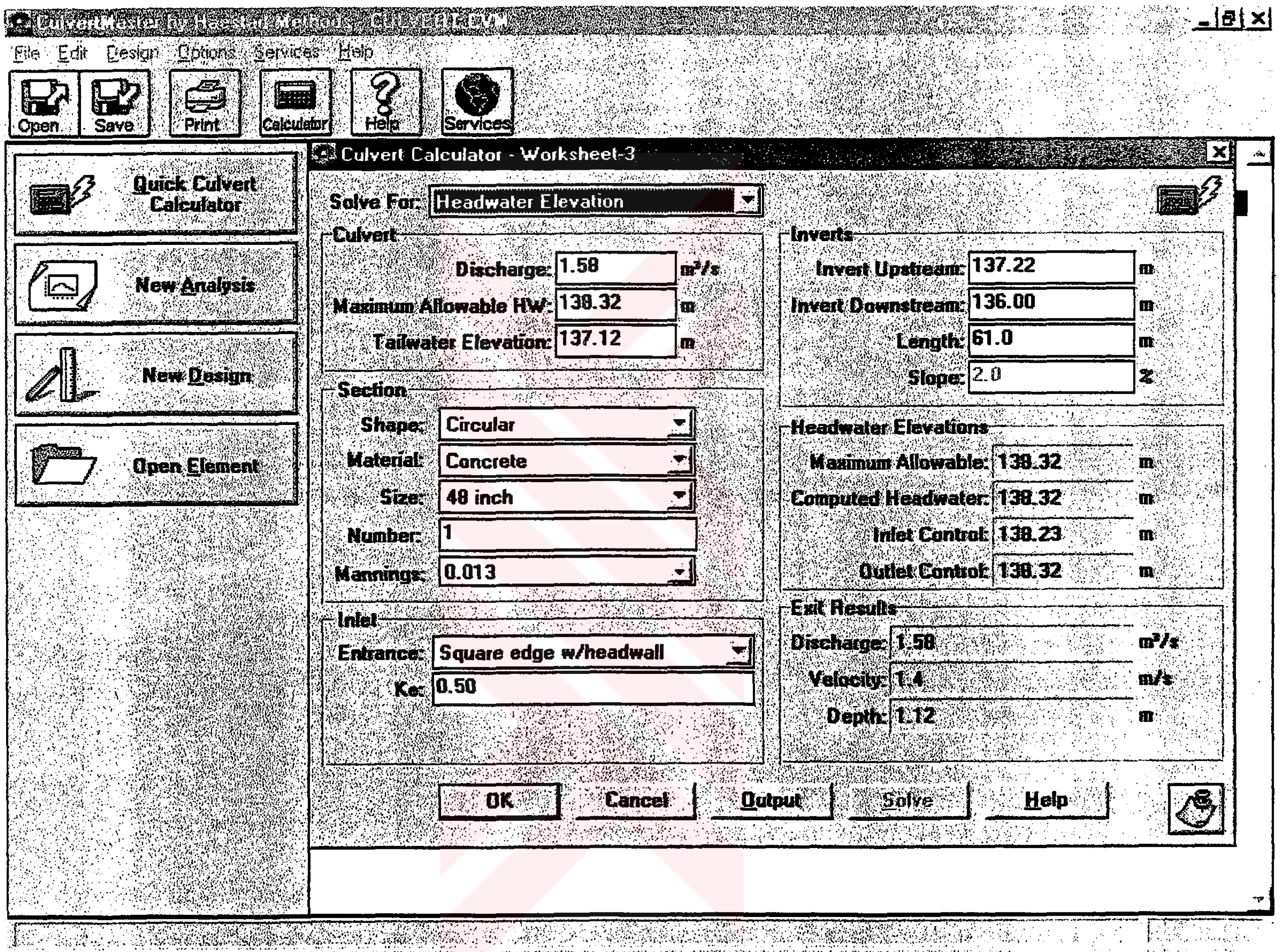

Figure C.19 CulvertMaster solution of illustrative example 2 (for $Q_d = 1.58\ m^3/sn$)

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	3.20	0.32 m³/s

Performance Curves Report
Worksheet-3

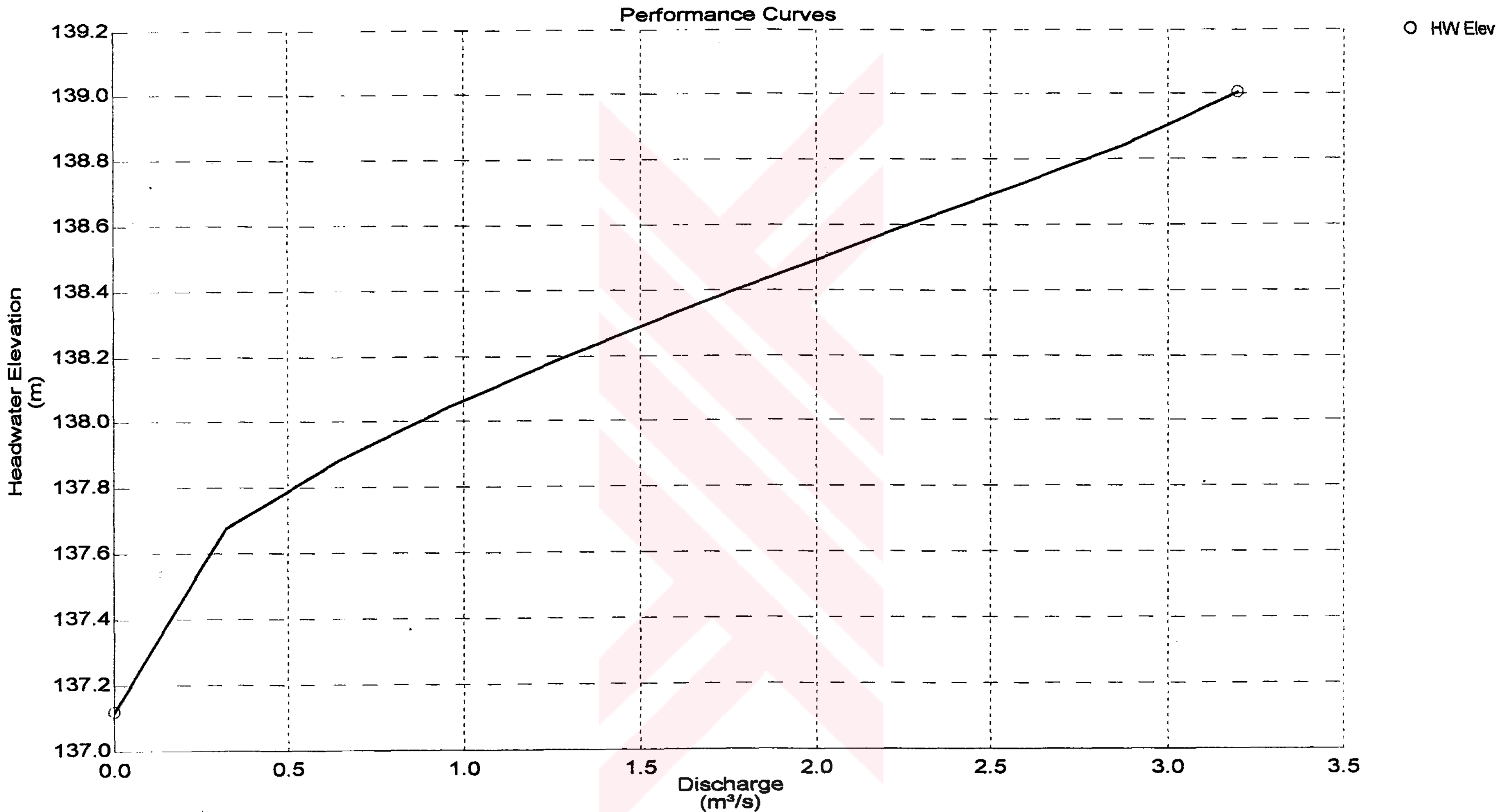

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Figure C.20 CulvertMaster performance curves report for $Q_d = 1.58\ m^3/sec$

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Culvert Calculator Report
Worksheet-3

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	138.32 m	Headwater Depth/ Height	0.91
Computed Headwater Elevation	138.32 m	Discharge	1.58 m^3/s
Inlet Control HW Elev	138.23 m	Tailwater Elevation	137.12 m
Outlet Control HW Elev	138.32 m	Control Type	Entrance Control

Grades			
Upstream Invert	137.22 m	Downstream Invert	136.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	CompositeS1S2	Depth, Downstream	1.12 m
Slope Type	Steep	Normal Depth	0.44 m
Flow Regime	N/A	Critical Depth	0.68 m
Velocity Downstream	1.4 m/s	Critical Slope	0.4 %

Section			
Section Shape	Circular	Mannings Coefficient	0.013
Section Material	Concrete	Span	4.00 ft
Section Size	48 inch	Rise	4.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	138.32 m	Upstream Velocity Head	0.28 m
Ke	0.50	Entrance Loss	0.14 m

Inlet Control Properties			
Inlet Control HW Elev	138.23 m	Flow Control	N/A
Inlet Type	Square edge w/headwall	Area Full	12.6 ft^2
K	0.00980	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	1
C	0.03980	Equation Form	1
Y	0.67000		

Figure C.21 CulvertMaster report for $Q_d = 1.58\ m^3/sec$

Project Title: Thesis Project
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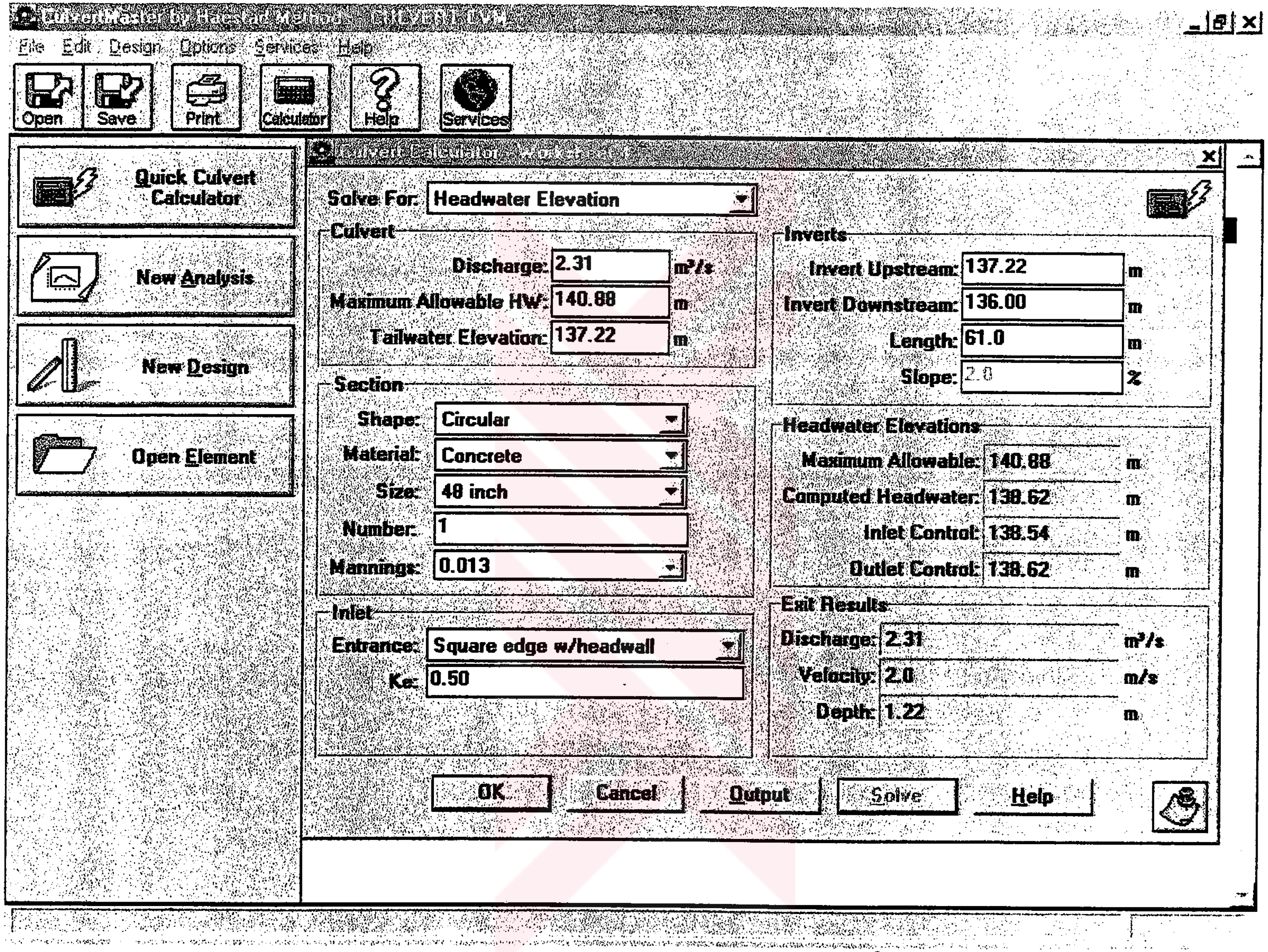

Figure C.22 CulvertMaster solution of illustrative example 2 (for $Q_d = 2.31\ m^3/sn$)

Performance Curves Report
Worksheet-4

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	5.00	0.50 m³/s

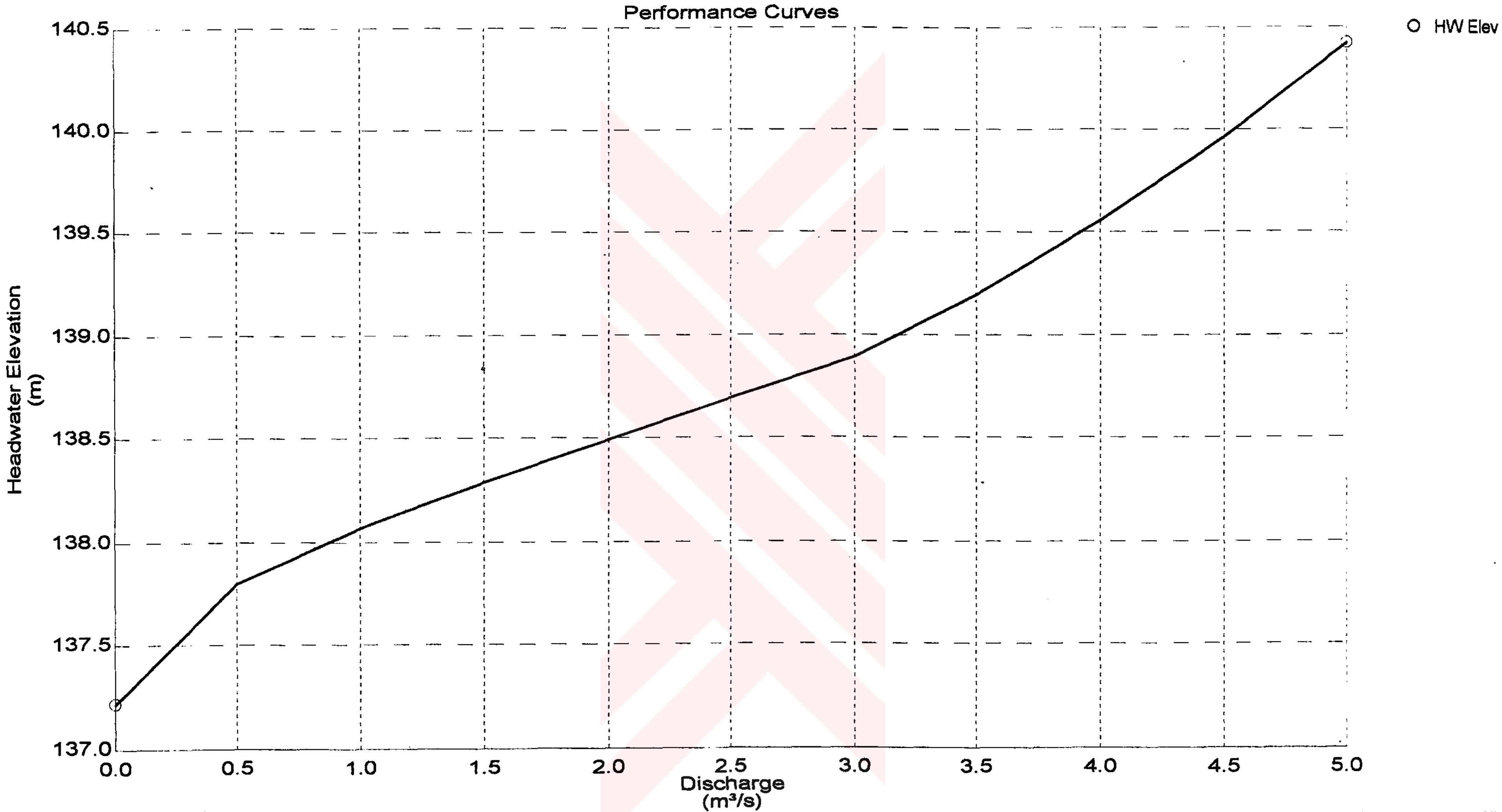

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Figure C.23 CulvertMaster performance curves report for $Q_d = 2.31\ m^3/sec$

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Culvert Calculator Report
Worksheet-4

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	140.88 m	Headwater Depth/ Height	1.15
Computed Headwater Elevation	138.62 m	Discharge	2.31 m^3/s
Inlet Control HW Elev	138.54 m	Tailwater Elevation	137.22 m
Outlet Control HW Elev	138.62 m	Control Type	Entrance Control

Grades			
Upstream Invert	137.22 m	Downstream Invert	136.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	CompositePressureS1S2	Depth, Downstream	1.22 m
Slope Type	N/A	Normal Depth	0.54 m
Flow Regime	N/A	Critical Depth	0.83 m
Velocity Downstream	2.0 m/s	Critical Slope	0.5 %

Section			
Section Shape	Circular	Mannings Coefficient	0.013
Section Material	Concrete	Span	4.00 ft
Section Size	48 inch	Rise	4.00 ft
Number Sections	1		

Outlet Control Properties			
Outlet Control HW Elev	138.62 m	Upstream Velocity Head	0.38 m
Ke	0.50	Entrance Loss	0.19 m

Inlet Control Properties			
Inlet Control HW Elev	138.54 m	Flow Control	N/A
Inlet Type	Square edge w/headwall	Area Full	12.6 ft^2
K	0.00980	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	1
C	0.03980	Equation Form	1
Y	0.67000		

Figure C.24 CulvertMaster report for $Q_d = 2.31$ m^3/sec

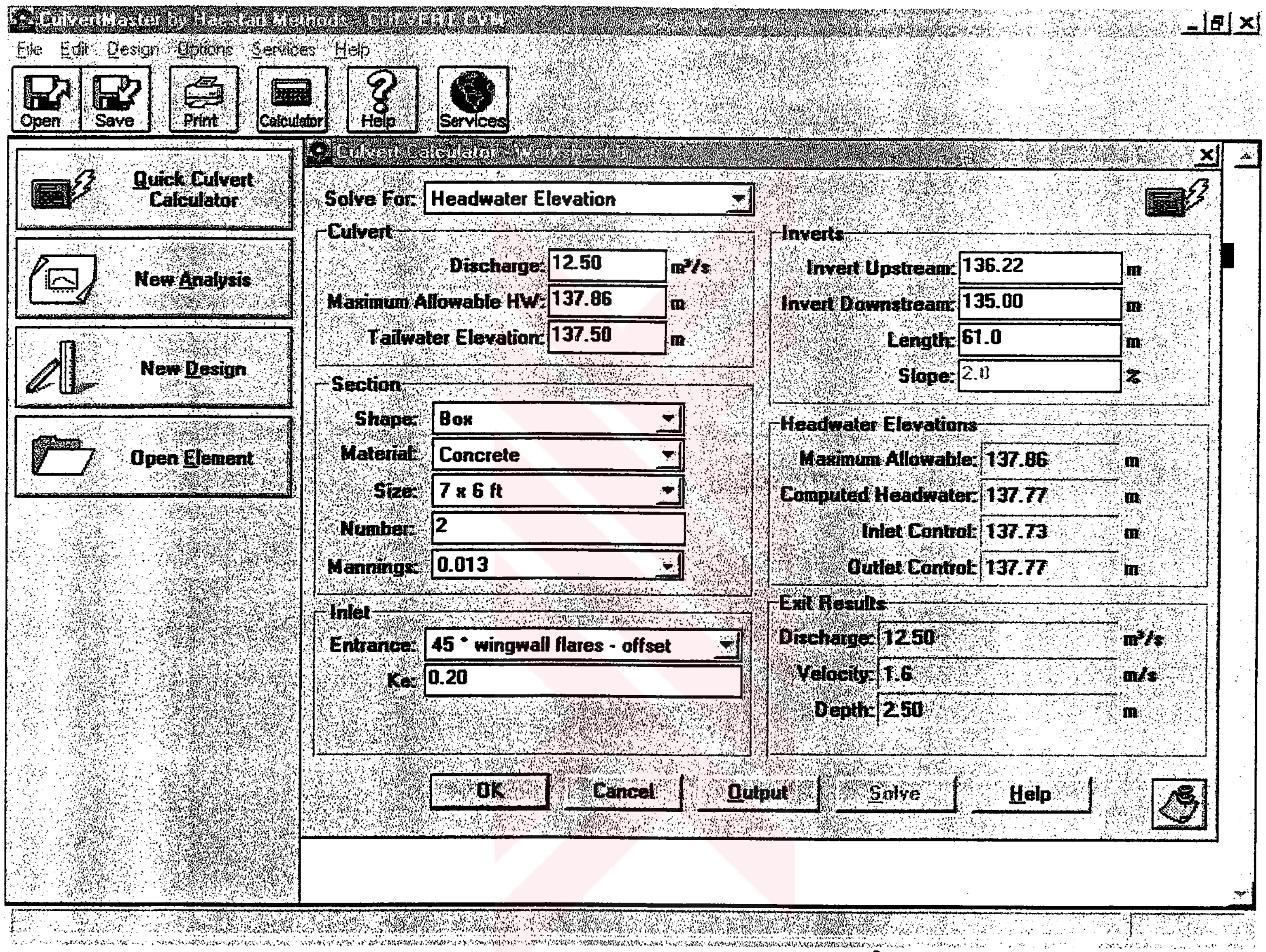

Figure C.25 CulvertMaster solution of illustrative example 3 (for $Q_d = 12.5\ m^3/sn$)

Performance Curves Report
Worksheet-5

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	25.00	2.50 m³/s

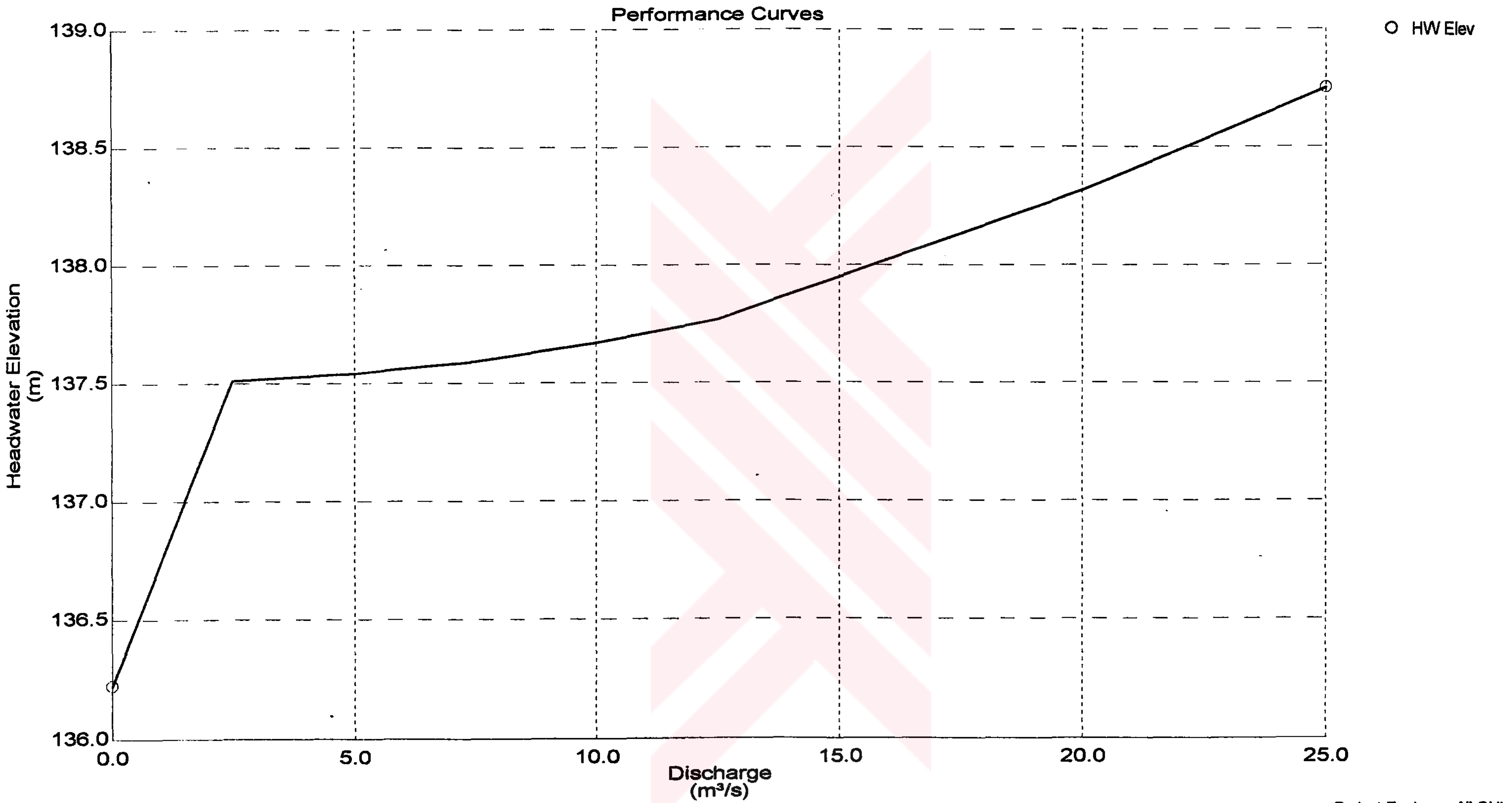

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Figure C.26 CulvertMaster performance curves report for $Q_d = 12.5$ m^3/sec

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Culvert Calculator Report
Worksheet-5

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	137.86 m	Headwater Depth/ Height	0.85
Computed Headwater Elevation	137.77 m	Discharge	12.50 m³/s
Inlet Control HW Elev	137.73 m	Tailwater Elevation	137.50 m
Outlet Control HW Elev	137.77 m	Control Type	Outlet Control

Grades			
Upstream Invert	136.22 m	Downstream Invert	135.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	CompositePressureS1	Depth, Downstream	2.50 m
Slope Type	N/A	Normal Depth	0.54 m
Flow Regime	Subcritical	Critical Depth	0.96 m
Velocity Downstream	1.6 m/s	Critical Slope	0.4 %

Section			
Section Shape	Box	Mannings Coefficient	0.013
Section Material	Concrete	Span	7.00 ft
Section Size	7 x 6 ft	Rise	6.00 ft
Number Sections	2		

Outlet Control Properties			
Outlet Control HW Elev	137.77 m	Upstream Velocity Head	0.33 m
Ke	0.20	Entrance Loss	0.07 m

Inlet Control Properties			
Inlet Control HW Elev	137.73 m	Flow Control	N/A
Inlet Type	45 ° wingwall flares - offset	Area Full	84.0 ft²
K	0.49700	HDS 5 Chart	13
M	0.66700	HDS 5 Scale	1
C	0.03020	Equation Form	2
Y	0.83500		

Figure C.27 CulvertMaster report for $Q_d = 12.5\ m^3/sec$

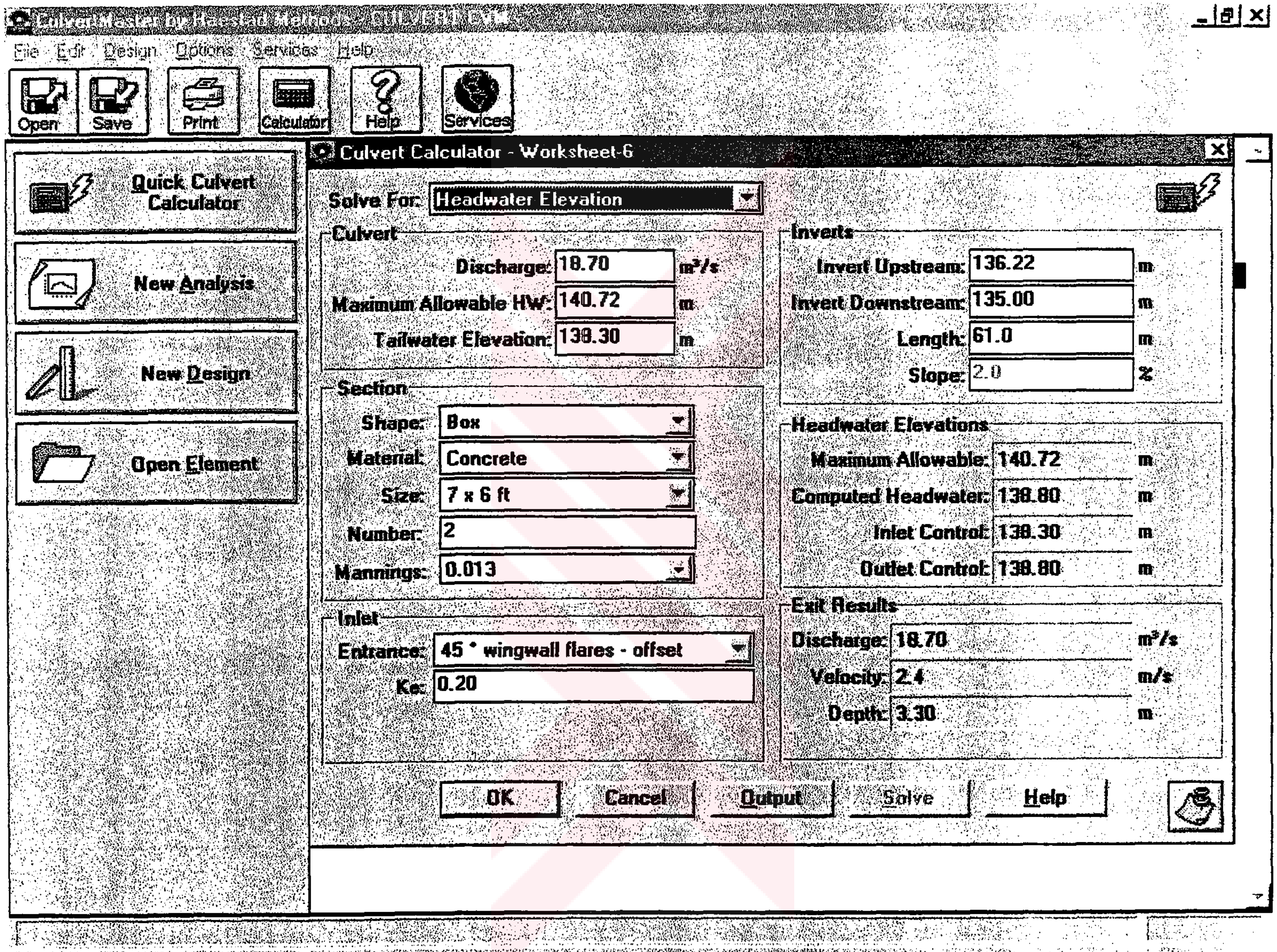

Figure C.28 CulvertMaster solution of illustrative example 3 (for $Q_d = 18.7\ m^3/sn$)

Performance Curves Report
Worksheet-6

Range Data:	Minimum	Maximum	Increment
Discharge	0.00	40.00	4.00 m³/s

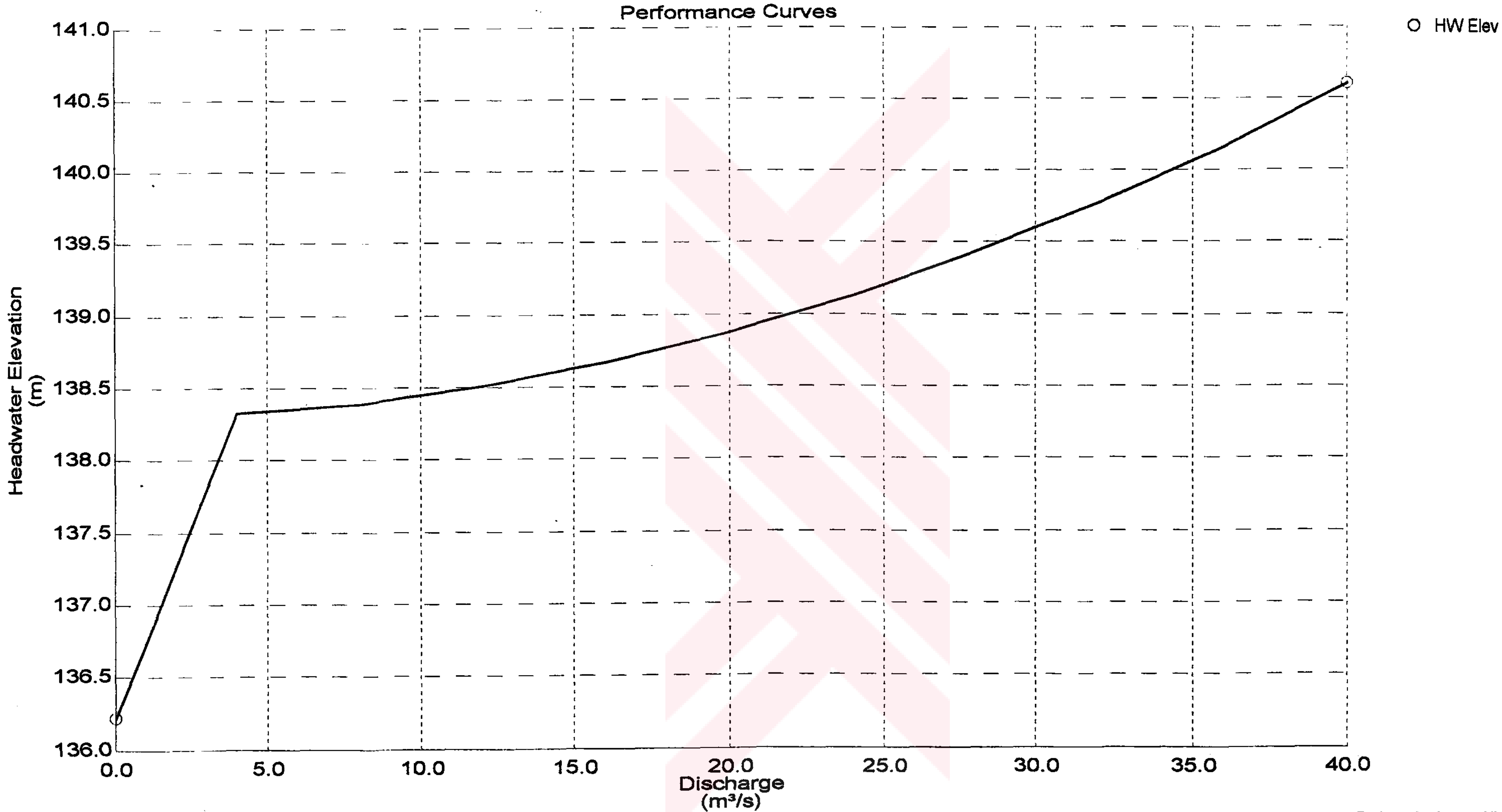

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Figure C.29 CulvertMaster performance curves report for $Q_d = 18.7\ m^3/sec$

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Culvert Calculator Report
Worksheet-6

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	140.72 m	Headwater Depth/ Height	1.41
Computed Headwater Elevation	138.80 m	Discharge	18.70 m^3/s
Inlet Control HW Elev	138.30 m	Tailwater Elevation	138.30 m
Outlet Control HW Elev	138.80 m	Control Type	Outlet Control

Grades			
Upstream Invert	136.22 m	Downstream Invert	135.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	Pressure	Depth, Downstream	3.30 m
Slope Type	N/A	Normal Depth	0.71 m
Flow Regime	N/A	Critical Depth	1.25 m
Velocity Downstream	2.4 m/s	Critical Slope	0.4 %

Section			
Section Shape	Box	Mannings Coefficient	0.013
Section Material	Concrete	Span	7.00 ft
Section Size	7 x 6 ft	Rise	6.00 ft
Number Sections	2		

Outlet Control Properties			
Outlet Control HW Elev	138.80 m	Upstream Velocity Head	0.29 m
Ke	0.20	Entrance Loss	0.06 m

Inlet Control Properties			
Inlet Control HW Elev	138.30 m	Flow Control	N/A
Inlet Type	45 ° wingwall flares - offset	Area Full	84.0 ft^2
K	0.49700	HDS 5 Chart	13
M	0.66700	HDS 5 Scale	1
C	0.03020	Equation Form	2
Y	0.83500		

Figure C.30 CulvertMaster report for $Q_d = 18.7\ m^3/sec$

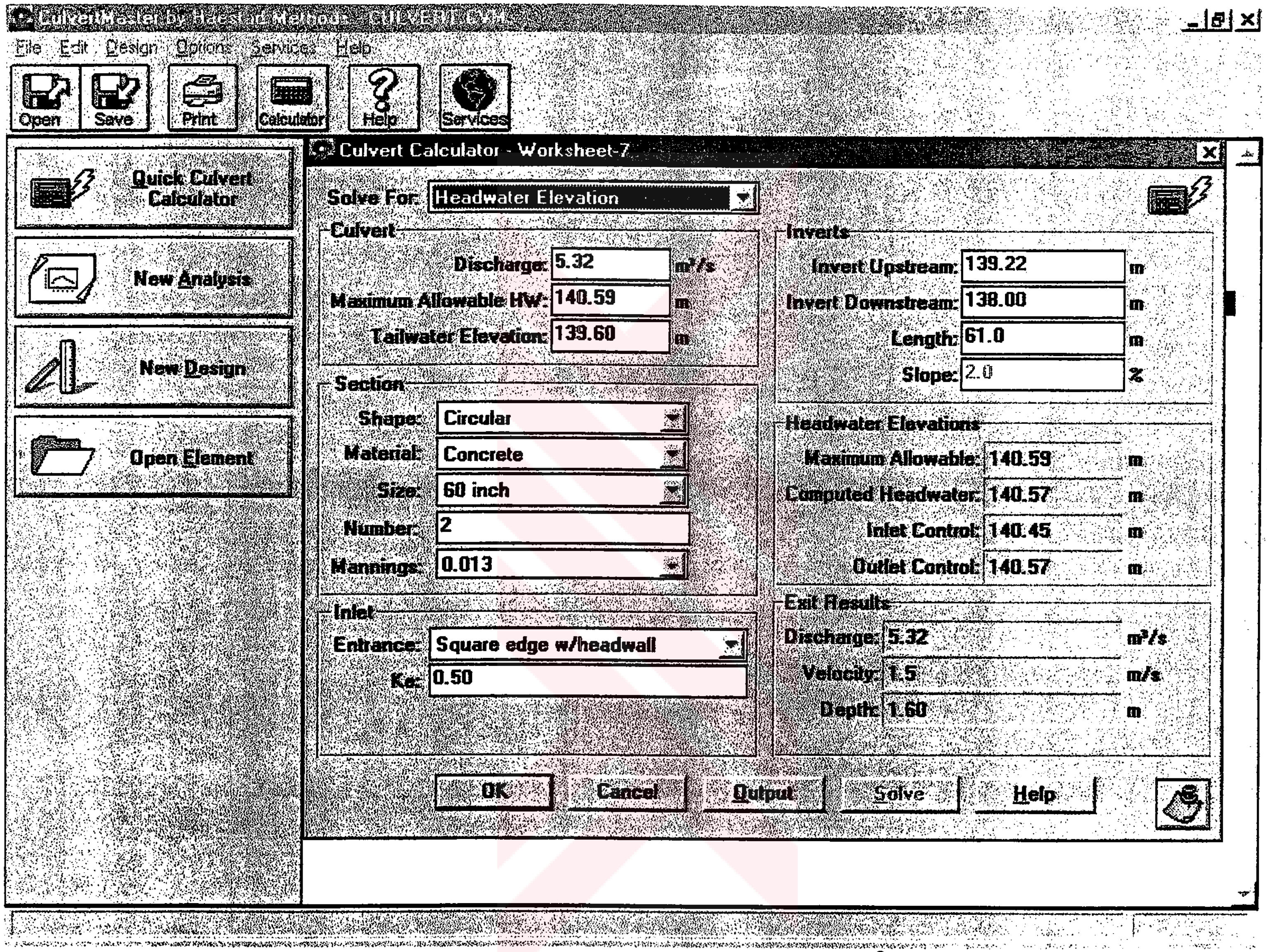
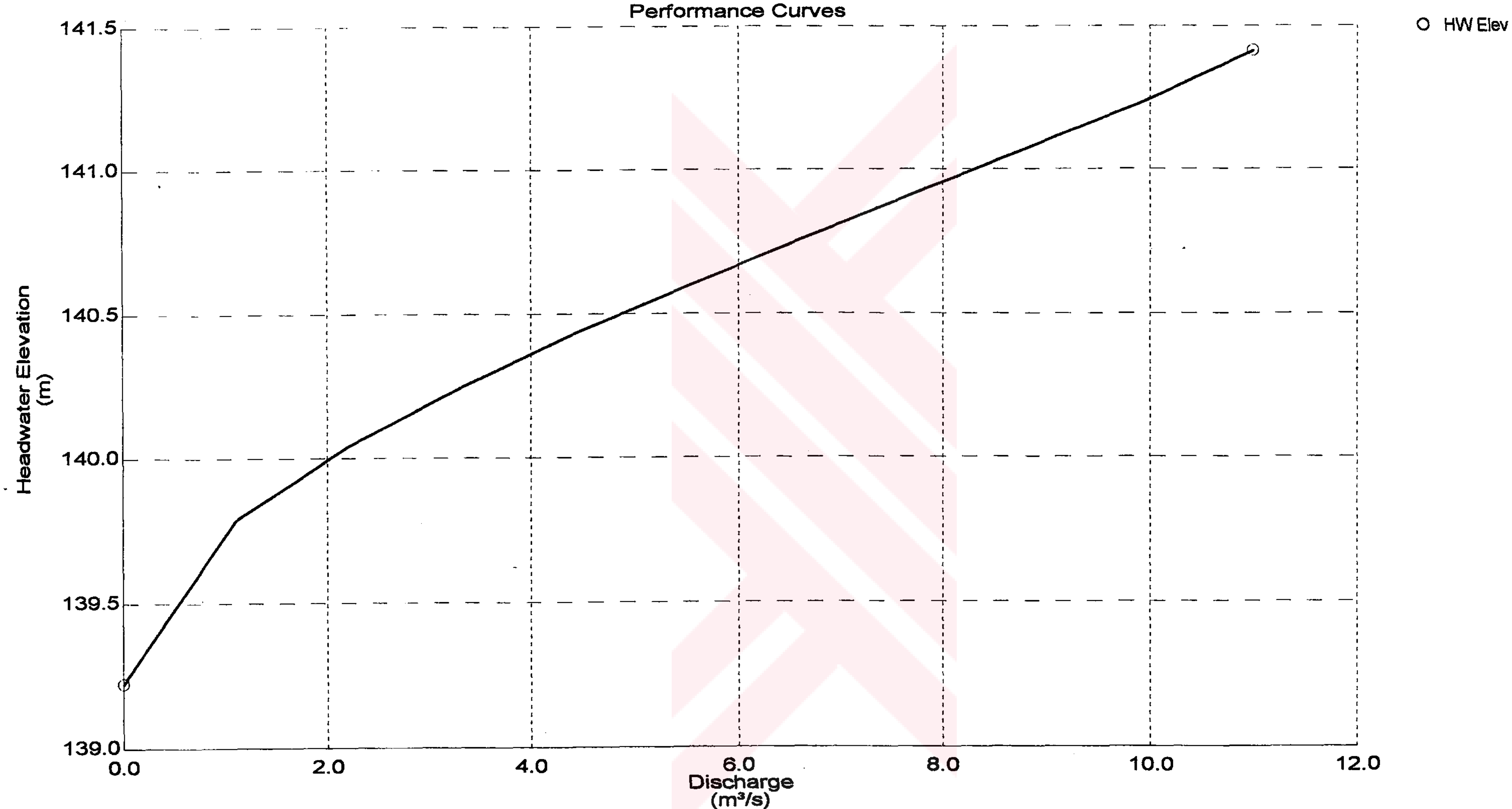


Figure C.31 CulvertMaster solution of illustrative example 4 (for $Q_d = 5.32\ m^3/sn$)

Performance Curves Report
Worksheet-7

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	11.00	1.10 m^3/s



Project Title: Thesis Project
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Figure C.32 CulvertMaster performance curves report for $Q_d = 5.32\ m^3/sec$

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Culvert Calculator Report
Worksheet-7

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	140.59 m	Headwater Depth/ Height	0.89
Computed Headwater Elevation	140.57 m	Discharge	5.32 m^3/s
Inlet Control HW Elev	140.45 m	Tailwater Elevation	139.60 m
Outlet Control HW Elev	140.57 m	Control Type	Entrance Control

Grades			
Upstream Invert	139.22 m	Downstream Invert	138.00 m
Length	61.0 m	Constructed Slope	2.0 %

Hydraulic Profile			
Profile	CompositePressureS1S2	Depth, Downstream	1.60 m
Slope Type	N/A	Normal Depth	0.52 m
Flow Regime	N/A	Critical Depth	0.84 m
Velocity Downstream	1.5 m/s	Critical Slope	0.4 %

Section			
Section Shape	Circular	Mannings Coefficient	0.013
Section Material	Concrete	Span	5.00 ft
Section Size	60 inch	Rise	5.00 ft
Number Sections	2		

Outlet Control Properties			
Outlet Control HW Elev	140.57 m	Upstream Velocity Head	0.34 m
Ke	0.50	Entrance Loss	0.17 m

Inlet Control Properties			
Inlet Control HW Elev	140.45 m	Flow Control	N/A
Inlet Type	Square edge w/headwall	Area Full	39.3 ft^2
K	0.00980	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	1
C	0.03980	Equation Form	1
Y	0.67000		

Figure C.33 CulvertMaster report for $Q_d = 5.32\ m^3/sec$

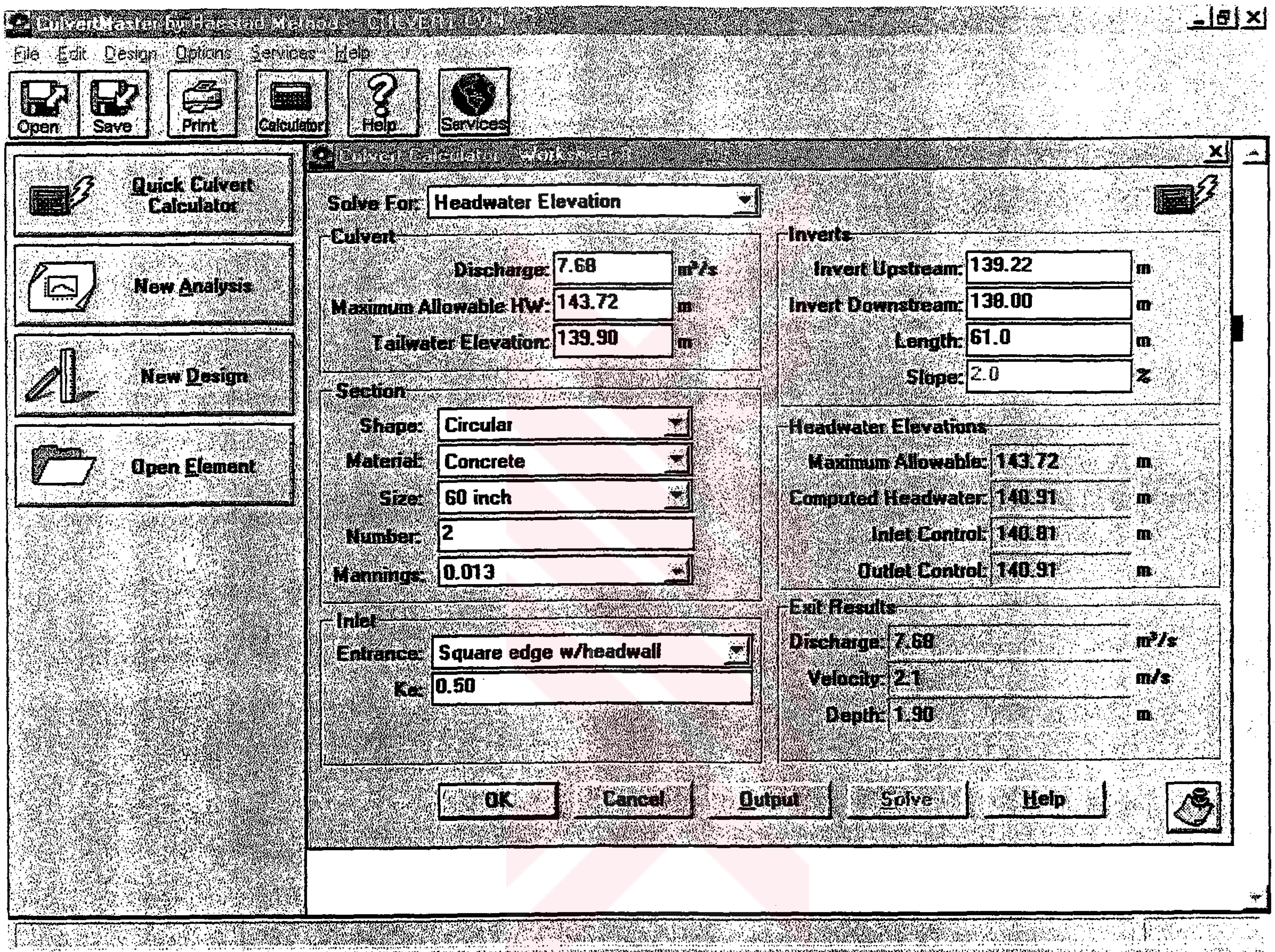
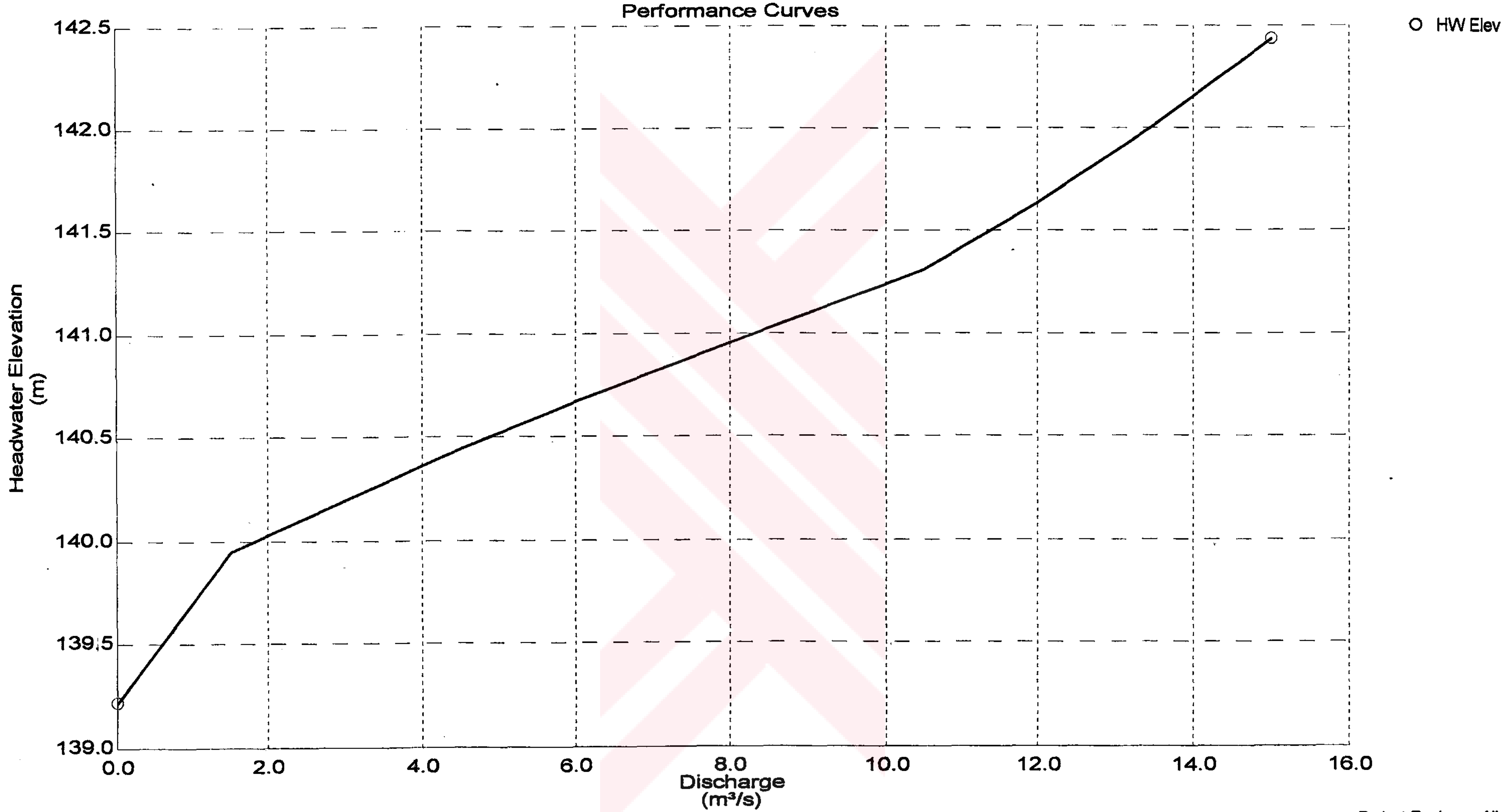


Figure C.34 CulvertMaster solution of illustrative example 4 (for $Q_d = 7.68\ m^3/sn$)

Performance Curves Report
Worksheet-8

Range Data:			
	Minimum	Maximum	Increment
Discharge	0.00	15.00	1.50 m³/s



Project Title: Thesis Project
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Figure C.35 CulvertMaster performance curves report for $Q_d = 7.68$ m^3/sec

Project Engineer: Ali GUL
CulvertMaster v1.0
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Culvert Calculator Report
Worksheet-8

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	143.72	m	Headwater Depth/ Height	1.11	
Computed Headwater Elevation	140.91	m	Discharge	7.68	m³/s
Inlet Control HW Elev	140.81	m	Tailwater Elevation	139.90	m
Outlet Control HW Elev	140.91	m	Control Type	Entrance Control	

Grades					
Upstream Invert	139.22	m	Downstream Invert	138.00	m
Length	61.0	m	Constructed Slope	2.0	%

Hydraulic Profile					
Profile	CompositePressureS1S2		Depth, Downstream	1.90	m
Slope Type	N/A		Normal Depth	0.64	m
Flow Regime	N/A		Critical Depth	1.02	m
Velocity Downstream	2.1	m/s	Critical Slope	0.4	%

Section				
Section Shape	Circular	Mannings Coefficient	0.013	
Section Material	Concrete	Span	5.00	ft
Section Size	60 inch	Rise	5.00	ft
Number Sections	2			

Outlet Control Properties					
Outlet Control HW Elev	140.91	m	Upstream Velocity Head	0.45	m
Ke	0.50		Entrance Loss	0.22	m

Inlet Control Properties					
Inlet Control HW Elev	140.81	m	Flow Control	N/A	
Inlet Type	Square edge w/headwall		Area Full	39.3	ft²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

Figure C.36 CulvertMaster report for $Q_d = 7.68\ m^3/sec$