CHAPTER 6

ANALYSIS OF FLOW NETS AT WEIRS, EMBANKMENTS AND GROUND WATER DILUTION SYSTEMS

(1) Weir on Sheet Piling in River-bed with Uniform Permeability

Seepage underneath sheet piling in a sandy river-bed is analysed in the diagram which follows. The sheet piling is so constructed that it dams up the river 3 ft. above bed level. Overflowing water is discharged on a heavy stone-in-wire mat on a suitable gravel base. The ground water flow net is constructed for the condition of surface flow occurring in the river and keeping the upstream sand body saturated.

The sand volume available for water storage will be increased somewhat by deposition upstream of the weir at a surface slope parallel to the original river-bed slope. The pump station which will benefit by the additional storage can be situated either in the immediate vicinity of the weir or some distance downstream.

A further effect of the weir is shown in Fig. 60. The ground water flow underneath the sheet piling is several times the seepage flow of the river due to the additional head produced by the weir. In the example chosen the ratio was found to be 10 : 1. One part of the seepage underneath the sheet piling is thus derived from normal river seepage and nine parts from fresh water infiltration from floods. The velocities are comparatively high (near the base of the deposits approximately 10 to 15 times the normal seepage velocities). The normal seepage flow is forced into a thin layer only a few feet thick over bed rock which is by no means as smooth in outline as in the theoretical case. All these factors will combine to bring about a mixing between the two types of water, i.e. dilution of ground water flow during floods. A further dilution can be brought about by the tube CDE, slotted at C and E, as explained in detail in Fig. 60. The pump station to benefit by ground water dilution is best situated some distance downstream of the weir.

The 3 ft. hydrostatic head is divided into eleven parts in the diagram. Potential lines are closest at the sheet piling and the hydraulic gradient at efflux next to the piling amounts to a drop of $\frac{3}{11}$ ft. in a distance of 3 ft., i.e. 1 : 11. Heave can be expected to occur at the critical hydraulic gradient $i_c = \frac{\gamma^1}{\gamma_w}$ [12]. If $\gamma^1 = 52$ lb. per cu. ft. and $\gamma_w = 62 \cdot 5$ lb. per cu. ft. then i_c is approximately unity. There is therefore a considerable factor of safety against piping by heave. Empirical rules for permissible seepage gradient under hydraulic structures specify 1 : 5 to 1 : 8 for sand [8 & 9]. D. W. Taylor[13] in discussing such empirical rules points out that fine sand is generally the safest foundation where failure due to piping is concerned. This is probably due to the uniformity of fine sand and the resulting uniformity in seepage velocities. Since the computed gradient of efflux of 1 : 11 is well below the permissible gradient quoted by various authorities there is no danger in the example analysed of failure due to undermining as a result of excessive seepage velocities.

The stone-in-wire mat should be long enough to act as an effective apron against scour. Dixey[10] in his *Handbook of Water Supply* suggests a length of apron of 8 ft. for a weir height of 3 ft. and for higher weirs a proportional increase in this length.

The stone-in-wire apron should rest on gravel or crushed rock which is large enough not to be washed out by floods. A mixture of sizes from $\frac{3}{4}$ in. to 3 in. is considered suitable for a nine-inch layer immediately below the apron (D_{15} of this coarse gravel layer will be, say 1 in.). In accordance with the rules given on page 72 the coarse gravel should be placed on a layer of finer material with $D_{85} = \frac{1}{4}$ of D_{15} of the coarse material $= \frac{1}{4}$ in. and a D_{15} size of say $\frac{1}{10}$th in. This lower layer should be approximately 6 in. thick and will seal off fine river sand (D_{85} equal to or greater than $\frac{1}{40}$th in.) against scouring by floods.

(2) Weir on Sheet Piling in River Bed with Variable Permeability

If the weir were constructed in a river-bed consisting of fine sand near the surface ($k = 58$) followed by a layer of medium sand ($k = 183$) and underlain by very coarse sand ($k = 1{,}580$) the flow net will assume the form shown in Fig. 61. The ratio between permeabilities of the three layers is approximately $\frac{1}{3} : 1 : 9$.

When the river-bed is completely saturated $h_1 = 3$ ft.

It will be seen that the flow net has been drawn with square fields in the zone $k = 183$ and rectangular fields in the other zones. In the fine sand zone the dimension of the rectangle in the direction of flow is one-third its width and in the zone of very coarse sand the length in the direction of flow is nine times the width. The reason for this construction is as follows:

The flow between any two flow lines $\frac{q_w}{N_f} = k\frac{\delta h_1}{\delta l} \times$ area of flow channel where δh_1 = loss of head in rectangle and δl = length of rectangle. For unit width of river-bed, $\frac{q_w}{N_f} = k\frac{\delta h_1}{\delta l} \times$ distance between flow lines

$$= k\frac{\text{width of rectangle}}{\text{length of rectangle}}\delta h_1$$

$\frac{q_w}{N_f}$ is constant and δh_1 is constant for equal pressure intervals

$$\therefore k\frac{\text{width of rectangle}}{\text{length of rectangle}} = \text{constant}$$

If, therefore, square fields are constructed in the central zone the fields in the other zones must have width to length ratios in inverse proportion to the value of k.

The following results are computed from the information given by the flow net:

$N_f = 14$

$N_d = 10$

q_w = underground flow at sheet piling $= kh_1\frac{N_f}{N_d}$ where k is the permeability in the zone with square fields $= 183 \times 3 \times \frac{14}{10} = 768$ cu. ft./day/unit channel width.

Normal ground water flow in coarse sand $= 1{,}580 \times 19 \times \frac{1}{300} = 100$ cu. ft./day/unit width.

Ejection tube PQ: The difference in head between P and Q will be $\frac{2\cdot5}{N_d}h_1 = \frac{2\cdot5}{10} \times 3 = 0\cdot75$ ft. Brack water will enter the slotted section at P and will be ejected at Q from where it will rise through the apron to be washed away by floods.

Injection tube XY: The difference in head between X and Y will be $\frac{5}{N_d}h_1 = 1\cdot5$ ft. Fresh water will enter the slotted section at X and will be injected into the lower ground water layers at Y causing dilution of the brack content of the latter.

(3) Sand Embankment on River-bed with Uniform Permeability

Where a suitable spillway can be provided on the side of the river, it is possible to achieve the storage plus ground water diluting effect by means of a sand embankment properly designed against overtopping by floods and washing out by seepage.

Fig. 62 illustrates such an embankment with adequate freeboard above high flood level. Seepage velocities will

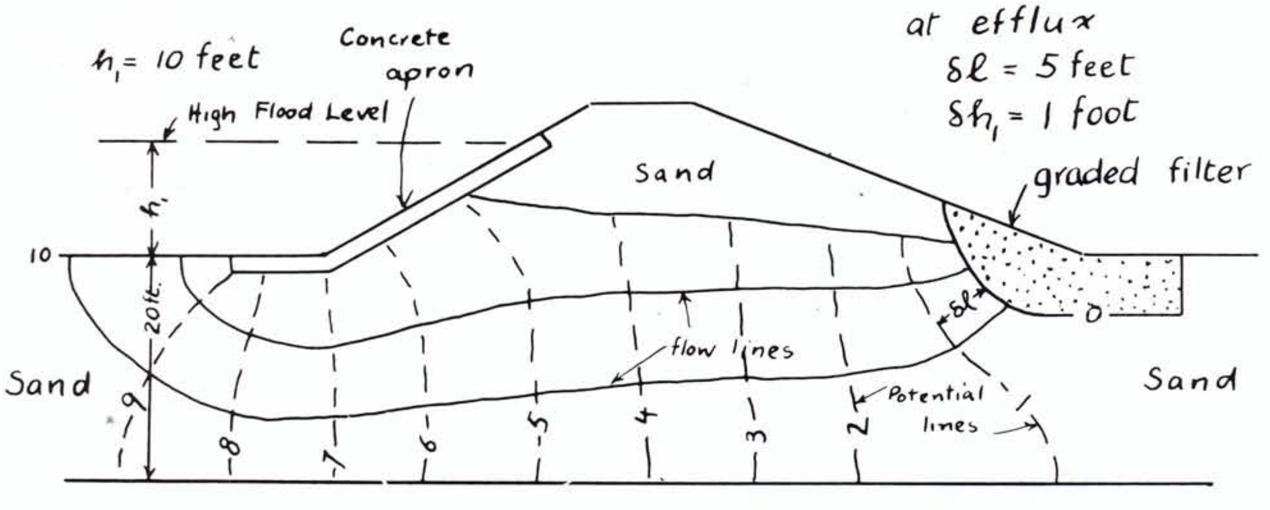

Fig. 62 Flow Net for Sand Embankment

Scale 0 1 2 3 4 5 10 feet

be greatest during high floods. In the example under consideration a concrete apron has been provided on the upstream side to lengthen the seepage path. The pressure gradient at efflux is $\delta h_1 = \frac{h_1}{10} = 1$ ft. in $\delta l = 5$ ft., which is the steepest gradient permitted in empirical rules (page 82).

As an additional factor of safety at efflux a graded filter is provided, which itself is not subject to uplift pressure by seepage emerging relatively slowly from finer material and which is therefore capable of counteracting heave of the finer material by means of its weight.

(4) Horizontal Drains in River-bed of Variable Permeability

Fig. 63 shows the flow net analysis for a sand filling 25 ft. deep with zones of fine, medium and very coarse sand similar to the Swakop River conditions (Fig. 32). The flow nets were drawn for conditions during floods and a loss of head from surface of sand to perforated drain pipe of 0·3 ft. Square fields are again adopted where k = 180 ft./day and rectangular fields with the appropriate width to length ratios in the other zones.

For the purpose of two-dimensional analysis an equivalent river-bed 260 ft. wide and of a uniform depth of 25 ft. was assumed.

The results of the analysis are as follows: Between surface of sand and perforated drain pipe $N_d = 3$ and $h_1 = 0 \cdot 3$ ft. Seepage between any two flow lines $= k \frac{h_1}{N_d} 260$ where k is the permeability in the region where square fields were adopted

$$= 180 \times \frac{0 \cdot 3}{3} \times 260$$

$$= 4{,}680 \text{ cu. ft./day}$$

According to Fig 63:
For a single drain $N_f = 17$, i.e. flow = 79,700 cu. ft./day.
For three drains, the first has

	N_f	=	16	i.e. flow	=	75,000 cu. ft./day
the second	N_f	=	13	i.e. flow	=	61,000 cu. ft./day
the third	N_f	=	14	i.e. flow	=	65,000 cu. ft./day
Total	N_f	=	43			201,000 cu. ft./day

The loss of head from the surface of the sand to the perforations for a total discharge of 201,000 cu. ft. a day through the three drains is therefore 0·3 ft. If the area drained is divided into twelve parts as in the tube well system (Fig. 39) and an amount $q = \frac{201{,}000}{12} = 16{,}175$ cu. ft./day be taken from each part then the loss of head for any other discharge can be taken as approximately $\frac{0 \cdot 3}{16{,}175} q = 0 \cdot 0000187q$.

Where each part is drained by one tube well instead of a transverse horizontal drain the loss of head was found to be approximately 0·0000485q or almost three times as great (Fig. 33). This result is to be expected as the seepage flow is drawn through much smaller cross-sectional areas of sand in the case of the tube well than in a horizontal drain extending over the full width of the sand.

Although it is not practicable to lay horizontal drains 22 ft. below river-bed level, the analysis of this theoretical case gives a general picture of the type of flow which may be expected with rows of tube wells (e.g. the distance up-stream and downstream from extraction points at which infiltration is induced, uniformity of infiltration rate, etc.).