data of the prototype situation and to describe the phenomenon by empirical relationships. Data available now enable us to evaluate the influence of various factors empirically in a more or less universal way. Attention is drawn to the great influence of the depth on the longitudinal salinity distribution in a river. Reference is made to a recent publication in the Journal of Hydraulic Research no. 4, 1968 by P. van der Burgh.

Design levels in the transition zone between the tidal reach and the river regime reach

J.W. van der Made,
Rijkswaterstraat, Netherlands

SUMMARY: A method for assessing design levels in those river reaches where the water stages are being influenced by both the river runoff and the sea level is described.

The ultimate decision depends on:

1. the life time of the structure to be protected;
2. the acceptable risk of flooding either by a storm surge at sea or by a flood wave on the river.
   The risk criteria for these two conditions may be different;
3. the frequency curves of both sea level and river discharges;
4. the dependency of the river stage at a given station in the transition zone on the sea level and on the river flow;
5. the possible correlation between sea level and river flow.

The method has been applied to the lower course of a river in the Netherlands.

RÉSUMÉ : Une méthode est présentée pour l'établissement des niveaux à prendre en considération pour le projet des ouvrages dans le cours inférieur d'un fleuve où les niveaux sont influencés par les débits d'amont ainsi que par les niveaux de la mer. En fixant ces niveaux de projet il convient de prendre en considération les points suivants :

1. la durée d'existence de l'ouvrage à protéger;
2. le risque acceptable d'une inondation de cet ouvrage par suite d'une marée-tempête ou d'une onde de crue. Les critères de ces deux cas peuvent être différents;
3. les courbes de fréquence des niveaux de la mer et des débits des crues;
4. l'influence du niveau de la mer et du débit du fleuve sur le niveau dans une station donnée de la zone de transition;
5. la corrélation qui peut exister entre le niveau de la mer et le débit du fleuve.

La méthode sera appliquée au cours inférieur d'un fleuve aux Pays-Bas.

I. ASSESSMENT OF DESIGN LEVELS OR DESIGN DISCHARGES
   USING FREQUENCY CURVES

The assessment of design levels for dikes, quay towns, embankments etc. along rivers or sea coasts should be based on the one hand on the lifetime of the object to be protected and on the other hand on the acceptable risk of destruction of the object. From these two
factors the frequency to be applied can be deduced. Then the design level is found by using the frequency curve, after extrapolation if necessary. A detailed description of this procedure has been given in a previous publication [1].

For river stations the use of discharges is preferable to the use of water levels, the latter being subject to influences of location and time. After assessment of the design discharge the corresponding design level is derived from the stage-discharge curve.

The frequency of water levels is expressed as the average number of times that a certain level is exceeded during one unit of time (e.g. one year). For a station on the sea coast with tidal movement the number of times means the number of high tides; for river station without tidal influence, it is the number of flood waves.

A second way to give frequencies of river discharges is by expressing them in the mean number of days that a certain discharge is exceeded in one time unit (year). This gives the total duration of exceedance but not the number of times the phenomenon (i.e. a flood wave) occurred.

In the Netherlands it is common practice to represent both frequencies in a single diagram. See figure 1 for the river Rhine at Lobith near the Dutch-German border. Here the values $E(Q)$ on the abcissa represent for the flood wave frequency curve ($T$) the mean number of flood waves per year exceeding a certain discharge.

On the same scale the values $D(Q)$ represent for the daily frequency curve ($D$) the mean number of days per year during which that discharge is exceeded. The ratio $n = D(Q)/E(Q)$ represents the mean duration that a given discharge was exceeded by a floodwave (days per flood wave).

![Figure 1. Exceedance frequency curves of the discharges of the river Rhine at Lobith](image-url)
II. SPECIFIC PROBLEMS IN THE TRANSITION ZONE

In the transition zone sea levels as well as river discharges influence the local water stages. Here the problems are rather complicated:

1. The interaction of the two factors hampers the extrapolation of the frequency curve. A given stage can result from a great number of combinations of sea level and river discharge. The degree of occurrence of all these combinations together determines the frequency of exceedance of the given stage.

2. A given frequency can have two meanings: "One time" may express "one high tide" as well as "one flood wave".

3. The safety standard for the tidal area may differ from that for the river area because of:
   (a) the contrast between salt sea water and fresh river water;
   (b) the fast rise of the water level during a storm surge along the coast in contrast with the relatively slow water movement in river floods;
   (c) the density of population and the economic value of the adjoining areas.

In the Netherlands the following frequencies of exceedance are considered as acceptable:
for the tidal area: $1.10^{-4}$ times per year;
for the river area: $3.10^{-4}$ times per year.
The key station for the tidal area is Hoek van Holland. The frequency curve for this station is represented in figure 2. For the river area the key station is Lobith (fig. 1).

III. TEST ON THE CORRELATION OF SEA LEVEL AND RIVER FLOW

The extent of occurrence of those combinations giving a certain stage in a station in the transition zone depends not only on the frequency distributions of the two phenomena sea level and river discharge, but also on the degree of correlation between them. This should be investigated first.
Figure 3. Exceedance frequency curves of the discharges of the river Rhine at Lobith (---10-year curves from fig. 1)

Figure 4. Exceedance frequency curve of the high tide levels in the winter of the North Sea at Hoek van Holland (---10-year curve from fig. 2.)
Both high storm surges and high flood waves, the relevant phenomena, occur mainly during the winter period, which lasts from November up to and including April. So it is preferable to apply only winter data instead of data for the whole year. In doing so we recognize a certain degree of correlation.

Thus the frequency curves for the winter periods only will be used. These curves are given in the figures 3 and 4.

It appears that only for the lower values of discharge and level there is a difference from the whole year curves as given in figure 1 and figure 2 respectively, which are drawn here like striped curves.

To investigate possible correlation between the two phenomena within the winter period the following test was applied to the conditions in the Netherlands. For those days when a storm surge occurred along the coast the river discharges were ordered and then plotted as a semi logarithmic frequency diagram. The positions of the points were compared with the mass curve of all the daily discharges in 60 winters.

It should be noted that a storm surge is defined as a high tide which exceeds a so-called "limit level". This level has a frequency of exceedance of 0.5 time per year. So on the average there are 0.5 "storm surges" per year. Because these floods occur in winter, this is the same as "0.5 storm surges per winter". Over an arbitrary period however the actual number of storm surges may differ slightly from the mean.

Figure 5: River Rhine discharge at stormflood days compared with all daily discharges

Figure 5 presents frequency curve of all the daily discharges of the river Rhine in winter during the period 1906-1965. This curve may be considered as the mass curve. The dots represent the river discharges on the days that storm surges occurred. During this period of 60 winters there have been 28 storm surges.
In addition to the mass curve the 2/3 control belt has been represented. The vertical distance between the mass curve and the control curves corresponds to the standard deviation of the discharges in a random sample.

On the basis of Bernoulli's law this distance amounts to:

\[ \sigma_q = \frac{1}{d(Q)/180} \sqrt{\frac{D(Q) \cdot \left\{ 1 - \frac{D(Q)}{180} \right\}}{N}} \]  

(1)

where:

- \( d(Q) \) the frequency distribution of the winter discharges (number of days per winter that the discharge is in a certain class interval);
- \( d(Q)/180 \) idem (fraction of time that the discharge is in a certain class interval);
- \( D(Q) \) the frequency of exceedance of the winter discharges (days per winter);
- \( D(Q)/180 \) idem (fraction of time);
- \( N \) the number of units in the sample.

For an arbitrary sample the value of \( N = 30 \) must be used, because on an average there are 30 storm floods in 60 winters.

It appears that only 6 out of the 28 storm surge discharges fall outside the control belt, from which only one flood plots at some distance from the belt limit line, whereas 5 floods fall upon that line. So the 28 discharges may be considered to be an ordinary random sample from the mass of the daily discharges.

From this the conclusion follows that for the conditions in the Rhine delta there is no correlation between the sea levels and the river discharges.

IV. ASSESSMENT OF FREQUENCIES OF EXCEEDANCE OF LEVELS UNDER THE INFLUENCE OF BOTH THE SEA AND THE RIVER

1. General discussion

A certain water level \( y \) at a station \( A \) situated in the transition zone can be reached or exceeded by many combinations of the river discharge \( Q \) and the sea level \( H \).

For the following calculations it is desirable to have a graph, giving for several water levels \( y \) at \( A \) the relation between the two basic factors \( Q \) and \( H \). These graphs will look like the diagrams in figure 6 for several stations along the Lower Rhine-Lek-Rotterdam Waterway in the Netherlands:

A. a station with only tidal effects : Hoek van Holland
B. a station where the tidal effects predominate : Streefkerk
C. a station with mixed tidal and river regimes : Schoonhoven (transition zone)
D. a station where the river regime predominates : Vreeswijk
E. a station with only river regime effects : Arnhem.

The method to be discussed is required only for stations in the transition zone, like type C. For stations of the types B and D one of the components dominates strongly, so that no very great errors are introduced by considering them as being purely tidal and purely river regime stations respectively.

Now consider a station of the type C. We are trying to find out the frequency of exceedance of the level \( y \). This level can be exceeded in different ways.

Figure 7 illustrates how, for instance, the 3 m level was exceeded in three actual periods. We distinguish the following types.
Figure 6. Curves of discharge-tide level combinations causing certain stages at gauge stations along the river lower Rhine-Lek-Rotterdam Waterway.
Figure 7. Types of high water levels at a station in the transition zone
Design levels in the transition zone between the tidal reach and the river regime reach

(a) Low discharges, high tides (e.g. February 1953)

It is not necessary here that a flood wave occurs on the river at all. When the discharge is relatively low we see a more or less varying discharge curve. The frequencies of exceedance can only reasonably be expressed in fractions of time, e.g. days per year or days per winter season.

The height of the stage $y$ at the station $A$ is principally caused by storms surges at sea with only a small contribution of the river discharge $Q$. The frequency of exceedance of the stage $y$ for discharges between $Q_1$ and $Q_1 + \Delta Q_1$ (see fig. 8) equals the frequency of exceedance of the corresponding high tide level $H$, during those discharges. Expressed as a formula:

$$E(y)_{Q_1 < Q < Q_1 + \Delta Q_1} = \left\{ \begin{array}{ll}
\text{time of occurrence of } Q_1 < Q < Q_1 + \Delta Q_1 \\
\text{frequency of exceedance of } H
\end{array} \right\}$$

$$= d(Q) \cdot \frac{E(H)}{180}$$

where:

$E(H)$ frequency of exceedance of the high tide $H$ in tides per winter period, and

$E(H)/180$ idem, in tides per day.

For all discharges between $Q = 0$ and $Q = Q_1$ the frequency of exceedance of $y$ amounts to:

$$E(y)_Q = E(y)_{Q < Q_1} = \sum_{Q=0}^{Q_1} d(Q) \cdot \frac{E(H)}{180}$$

This includes the total frequency of all cases situated in area I in figure 8.

If the curve $(Q, H)$ in figure 8 is horizontal over the whole diagram, which is the case in a pure tidal station, where $y$ does not depend on the runoff at all, formula (3)
transfroms to:

$$E(y) = \frac{E(H)}{180} \sum_{q=0}^{\infty} d(q) = \frac{E(H)}{180} \cdot 180 = E(H)$$  \hspace{1cm} (4)$$

Then the frequencies of the high tides at sea and the corresponding stages on the river are the same.

(b) Medium discharges, medium high tides \hspace{1cm} (e.g. December 1965, fig. 7)

The discharge $Q_1$ should be choosen at a point where the stage $y$ is clearly influenced by both basic factors. As will appear the value of $Q_1$ may be rather arbitrary for the higher values of the stage $y$ discussed in this paper.

Assume the discharge $Q_1$ to be in the area of the medium flood peaks. Let $E(Q_1)$ be the frequency of exceedance of $Q_1$ in flood waves per winter, whereas the discharge $Q_1$ is exceeded during $n$ days per flood wave on an average. To exceed the level $y$ at $A$, a tide equal to or higher than $H_1$ is needed. Only those flood waves should be taken into account the peaks of which coincide with one or more sufficient high tides.

The level $H_1$ at the sea coast be exceeded during $E(H_1)$ tides per winter on an average. This equals $E(H)/180$ tides per day and approximately $E(H)/360$ tides per tidal period. The chance that the level $H_1$ be exceeded once or more in a period of $n$ days or $2n$ tides, amounts according Bernoulli's law, to:

$$p(H_1) = 1 - \left(1 - \frac{E(H_1)}{360}\right)^{2n}$$  \hspace{1cm} (5)$$

Consequently the frequency of exceedance of the level $y$ will be for the case under consideration:

$$E(y)_{H_1} = E(Q_1) \cdot p(H_1)$$

$$= E(Q_1) \cdot \left[1 - \left(1 - \frac{E(H_1)}{360}\right)^{2n}\right]$$  \hspace{1cm} (6)$$

If $E(H)$ is small enough, this can be approximated by

$$E(y)_{H_1} = E(Q_1) \cdot \frac{n}{180} \cdot E(H_1) = D(Q_1) \cdot \frac{E(H_1)}{180}$$  \hspace{1cm} (7)$$

This means that for tides with low frequencies of exceedance the probability of occurrence during a flood wave equals the frequency of exceedance in periods of $n$ days. Then the cases considered here (area II) join the cases in area I for low river discharges.

The question may be raised to what point of the curve $(Q_1, H_1)$ one may go without introducing an unacceptable difference. Therefore we should calculate the difference between the frequency of exceedance and the probability of the occurrence of $H$. In formula:

$$\delta = \frac{n}{180} \cdot E(H_1) - \left[1 - \left(1 - \frac{E(H_1)}{360}\right)^{2n}\right]$$  \hspace{1cm} (8)$$

A series development of the second term leads to:

$$\delta = n(2n-1) \cdot \frac{E(H_1)^2}{360}$$  \hspace{1cm} (9)$$
Consider the case that the discharges have an average duration of exceedance of \( n = 4 \) days per flood wave. If the difference is not allowed to exceed the value 0.01 then from (9) follows for the maximum allowed frequency of exceedance of \( H_1 \) the value:

\[
E(H_1) \text{ max} = 6.8 \text{ tides per winter.}
\]

The corresponding high tide level is \( NAP + 1.10 \) m. Apparently this is a relatively low value. It should be emphasized however that this is an ultimate lowest limit for \( H \). One is always free to fix the point \( (Q_1, H_1) \) at a higher value of \( H \).

A lowest limit for the discharges does not exist at all, but for very low discharges the results of the calculations will not be very accurate if a very small value of \( Q \) is chosen for \( Q_1 \).

The best way is to choose the point \( (Q_1, H_1) \) somewhere half way along the curve.

\((c)\) High discharges, low high tides \( \text{(e.g. January 1926, fig. 7)}\)

Discharges, higher than \( Q_1 \) can cause a stage \( y \) at station \( A \) if combined with lower high tides than \( H_1 \).

Following the procedure in the case \( b) \) for a discharge \( Q_1 + A Q_1 \) we can derive:

\[
E(y)_{\text{max}} = E(Q_1 + A Q_1) \cdot p(H_1 - A H_1)
\]

To avoid duplication we must subtract however: (see fig. 7)

\[
E(Q_1 + A Q_1) \cdot p(H_1).
\]

because the combinations \( (Q > Q_1 + A Q_1, H > H_1) \) were included in the case \( (b) \).

To the frequency of \( y \) only contributes:

\[
\Delta E(y) = E(Q_1 + A Q_1) \cdot \{ p(H_1 - A H_1) - p(H_1) \}
\]

which leads to a total contribution of the higher discharges:

\[
E(y)_{\text{max}} = \sum_{H = H_1}^{H = H_0} E(Q) \cdot A p(H)
\]

These cases are all included in the area III.

The difference \( A p(H) \) represents the difference between the probability values of \( H \) and \( H - A H \), which levels correspond respectively with discharges \( Q_h \) and \( Q_h + A Q_h \) in giving the same level \( y \) at the station \( A \).

It will be clear, that for the lower values of \( H \) the real probability value \( p(H) \) from formula (5) should be used and not the average frequency value in an \( n \)-days period. A high flood wave namely, during which a certain level is exceeded by several relative low tides (fig. 7: the 3 m level in January 1926) is considered to give a single excess, although there may be some interruptions.

For practical reasons however it is preferable to work also in the area III with the frequency of exceedance of the daily discharges \( D(Q) \). Then the following formula, derived from (13), should be used:

\[
E(y)_{\text{max}} = \sum_{H = H_0}^{H = H_1} D(Q) \cdot \frac{A p(H)}{n}
\]
The value $\Delta p(H)/n$ is the chance on exceedance of $H$ in an arbitrary period of $n$ days, divided by $n$ days.

If the stage $y$ is fully independent on the sea level, which is the case for a pure river station the $(Q, H)$ relation has a vertical trend. Then formula (13) transforms to:

$$E(y) = E(Q) \sum_{H=H_0}^{\infty} p(H) = E(Q)$$  \hspace{1cm} (15)$$

Then the frequencies of exceedance of the river discharges and those of the corresponding stages $y$ have the same values.

Summarizing: the total frequency of exceedance of the level $y$ equals the sum of frequencies of exceedance of the three cases considered above. In formula:

$$E(y) = E(y)_t + E(y)_r + E(y)_s$$  \hspace{1cm} (16)$$

---

**Figure 9.** Diagram for derivation the frequency of exceedance $E(y)$: $E(y) = E(y)_t + E(y)_r + E(y)_s$
2. DERIVATION OF THE EXCESS FREQUENCIES $E(y)$

To derive the frequencies of exceedance $E(y)$ for several levels $y$ at the station $A$ the following procedure can be applied.

Consider a four quadrant diagram of a composition as is given in figure 9. In quadrant $B$ the exceedance frequency curve of the daily discharges $D(Q)$ has been represented; in quadrant $D$ the exceedance frequency curve of the high tides $E(H)/180$ is given as well as the probability curve $p(H)/n$. The curves in these two quadrants can be used for each station in the transition zone.

Quadrant $C$ represents the locus of all $(Q, H)$ combinations that cause a certain stage $y$ at the station $A$. Diagrams of this type were already represented in the figures 6 and 8. For each value of $y$ a $(Q, H)$ curve can be drawn.

Each $(Q, H)$ curve in quadrant $C$ can, with the aid of the curves in the quadrants $B$ and $D$ be projected into quadrant $A$. The resulting curve there gives the relation between the factors (frequencies, probabilities) that should be multiplied in the different cases to find the frequency of exceedance of $y$.

In quadrant $A$ we find back the three areas mentioned before.

In fact these three areas in quadrant $A$ must be integrated, taking into consideration the logarithmic scale. The total area $(I+II+III)$ yields the frequency of exceedance of the level $y$.

After assessment of the frequencies of exceedance of several values of $y$ the exceedance frequency curve of the water levels at the station $A$ can be composed. The result for the station Schoonhoven at the river Lek is given in figure 10. As appears the calculated curve for the high levels joins reasonably the curve of the lower levels, which has been derived from observations.

V. ASSESSMENT OF THE DESIGN LEVEL

When the safety standard of flooding has the same value for both the tidal and the river regime regions the design level is easily found on the frequency curve like figure 10.

When different standards are in force some correction is needed. Then the following procedure can be applied.

Suppose a risk criterion of "$R$ flood waves per winter" be accepted for the upper river and a criterion of "$S$ storms surges per winter" for the coast and the tidal river reaches.

The value "$R$ excesses" be corresponding with a discharge of $Q_R$ m$^3$/s, whereas the discharge of $Q_s$ m$^3$/s be exceeded by $S$ flood waves per year. Next we multiply the whole scale of discharges by the ratio $Q_R/Q_s$. Then a new scale of discharge values $Q^*$ (see fig. 11) originates, for which holds:

\[ Q^* = \frac{Q_R}{Q_s} \cdot Q \]  \hspace{1cm} (17)

This means that we reduce the actual discharges to lower values. Let at the same time the safety requirements for the river be tightened up, namely from $R$ up to $S$ excesses per year. This ultimately gives no changes for the design discharge being in the first case $Q_R$ and the second case:

\[ Q_s^* = \frac{Q_R}{Q_s} \cdot Q_s = Q_R \]  \hspace{1cm} (18)

 Consequently we combine the reduced discharges $Q^*$ with the actual sea stages and apply a general safety standard of $S$ excesses per winter. With the reduced discharges $Q^*$ the procedure, described in par. IV can be followed. This yields an imaginary frequency curve for the levels $y$ at the station $A$. The level, corresponding with the general risk criterion of $S$ excesses per winter is assessed to be the local design level. It is interesting,
**Figure 10.** Exceedance frequency curve Schoonhoven (1941-1960)

**Figure 11.** Reductions of the river discharges for equalizing the safety standards
of course, to know the real frequencies of exceedance of the levels at the station A. These are found by applying the original discharges Q in the procedure described.

In particular the frequency of exceedance of the design level can be deduced. This will be a transition value \( T \), somewhere between the above values \( R \), respectively \( S \) excesses per year.

The nearer the station \( A \) is situated to the coast, the more the value \( T \) approaches to the value \( S \). Conversely, the more the station is situated inland, the closer will be the values of \( T \) and \( R \).

From the value \( T \) we can derive to what extent a given station will be a tidal station or a river station. Suppose the station \( A \) to be a \((d. 100\%)\) tidal station and a \((1-d)\) 100\% river station.

Then the value \( T \) amounts to:

\[
T = d \cdot S + (1 - d) R
\]

(19)

from which follows:

\[
d = \frac{R - T}{R - S}
\]

(20)

In figure 10 the reduced frequency curve has been drawn. The design level is found on this curve for a frequency of exceedance of \( S = 1 \times 10^{-4} \) times per year.

This level, projected on the original frequency curve delivers a frequency of exceedance of \( T = 2 \times 10^{-4} \) times per year.

Memorizing that for the river region holds \( R = 3 \times 10^{-4} \) times per year we find a tidal influence at that station of

\[
d \cdot 100\% = \frac{3 - 2}{3 - 1} \times 100\% = 50\%
\]

In figure 12 is given the longitudinal profile of the design levels along the rivers Lek-Rotterdam Waterway in respectively the tidal zone, the transition zone and the lower end of the river regime zone. The safety standards are also given in this diagram.

This diagram indicates where both factors, sea level and river discharges, exercise their influence.

In the tidal zone the storm surges dominate. In fig. 12 the curve for the design storm surge stages has been extrapolated in to the inland direction. This represents the case, in which the design storm coincides with a medium river runoff.

Another extrapolation has been given for the level in the tidal zone, caused by a design flood during a medium tide. Only, in the transition zone both flood waves and storm surges cause high stages. The resulting curve for the design levels in that zone has also been represented in figure 12.

**VI. FINAL REMARKS**

The numerical values of the results refer to the situation before the execution of the delta works in the south west of the Netherlands. Although the Rotterdam Waterway will not be closed off, these works will cause some alterations to the waterlevels along that river because of the interaction with the southern river branches which are to be dammed off.

In the above example of the northern Rhine branch, the river Lower Rhine-Lek-Rotterdam Waterway was dealt with. For the other Rhine branches as well as for the other river in the Netherlands, the river Meuse, diagrams like figure 12 are in course of preparation.
Figure 12. Design levels and safety standards along a river reach

Principally this diagram can be constructed for any lower river in the world if the two frequency curves, related to the sea water stages and to the river discharges respectively, and the influence of these two phenomena on the local water stages along that river are known.

REFERENCES