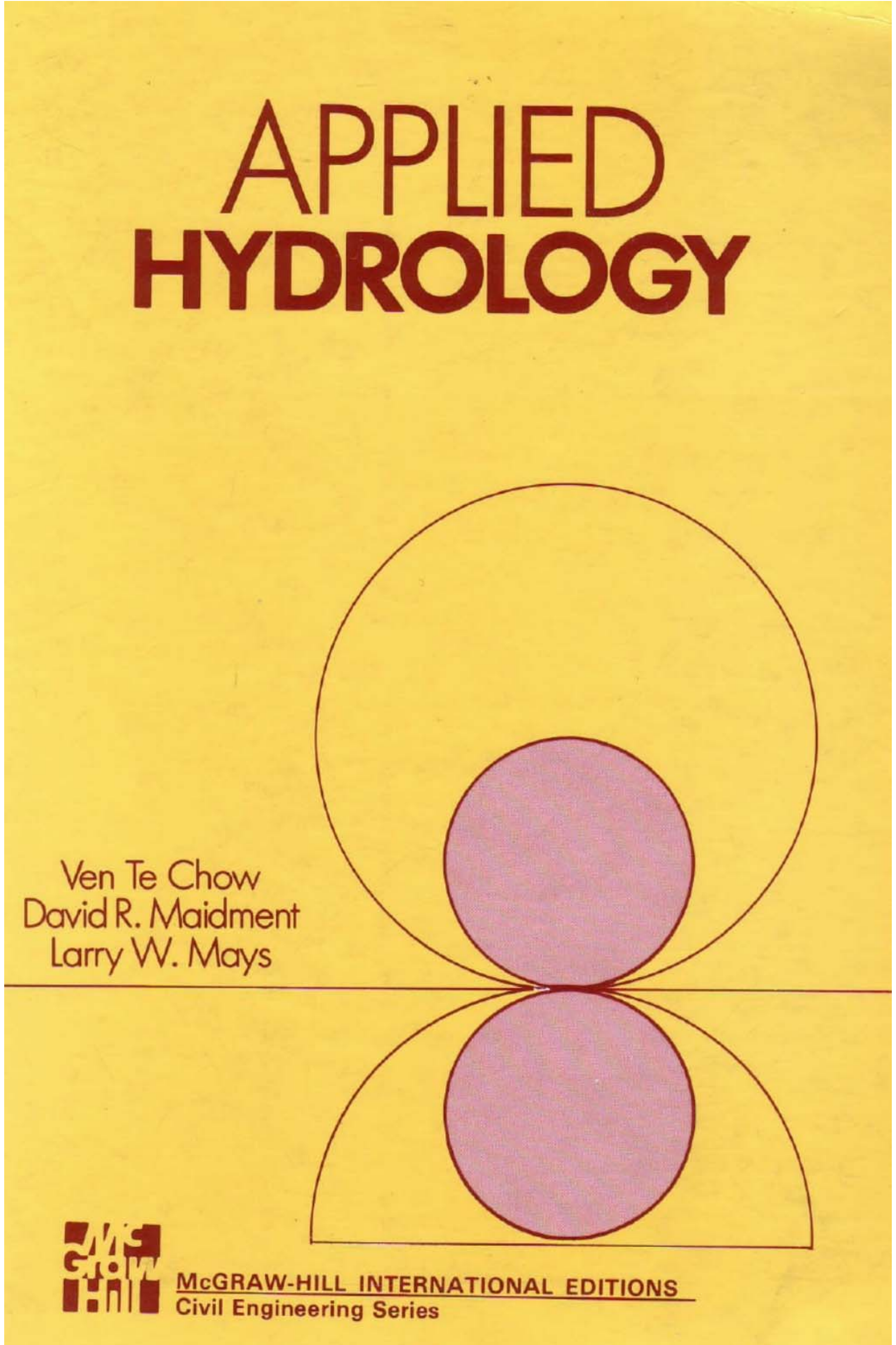



CHAPTER
13

# HYDROLOGIC DESIGN

**Hydrologic design is the process of assessing the impact of hydrologic events on a water resource system and choosing values for the key variables of the system so that it will perform adequately.**

**Hydrologic design may be used to develop plans for a new structure, or to develop management programs for better control of an existing system.**

The purposes of water resources planning and management may be grouped roughly into two categories.

1. *water control,* (drainage, flood control, pollution abatement, insect control, sediment control, and salinity control)
2. *water use* and management, (domestic and industrial water supply, irrigation, hydropower generation, recreation, fish and wildlife improvement, low-flow augmentation for water quality management, and watershed management).

In either case, **the task of the hydrologist** is the same, namely,
**to determine a design inflow, to route the flow through the system, and to check whether the output values are satisfactory**.

The difference between the two cases is that

design for water control is usually concerned with **extreme events of short duration**, such as the instantaneous peak discharge during a flood, or the minimum flow over a period of a few days during a dry period, while

design for water use is concerned with the **complete flow hydrograph over a period of years**.

The **optimal magnitude for design** is one that balances the conflicting considerations of **cost and safety**

The **hydrologic design scale** is the range in magnitude of the design variable (such as the design discharge) within which a value must be selected to determine the inflow to the system

Its **probabilities of occurrence** can be estimated adequately when **hydrologic records of sufficient length** are available for **frequency analysis**.

The probabilistic approach is less subjective and more theoretically manageable than the deterministic approach.

**Probabilistic methods** also lead to logical ways of determining optimum design levels, such as by **hydroeconomic and risk analyses**

**The *estimated limiting value* (ELV)** is defined as *the largest magnitude possible for a hydrologic event at a given location, based on the best available hydrologic information.*

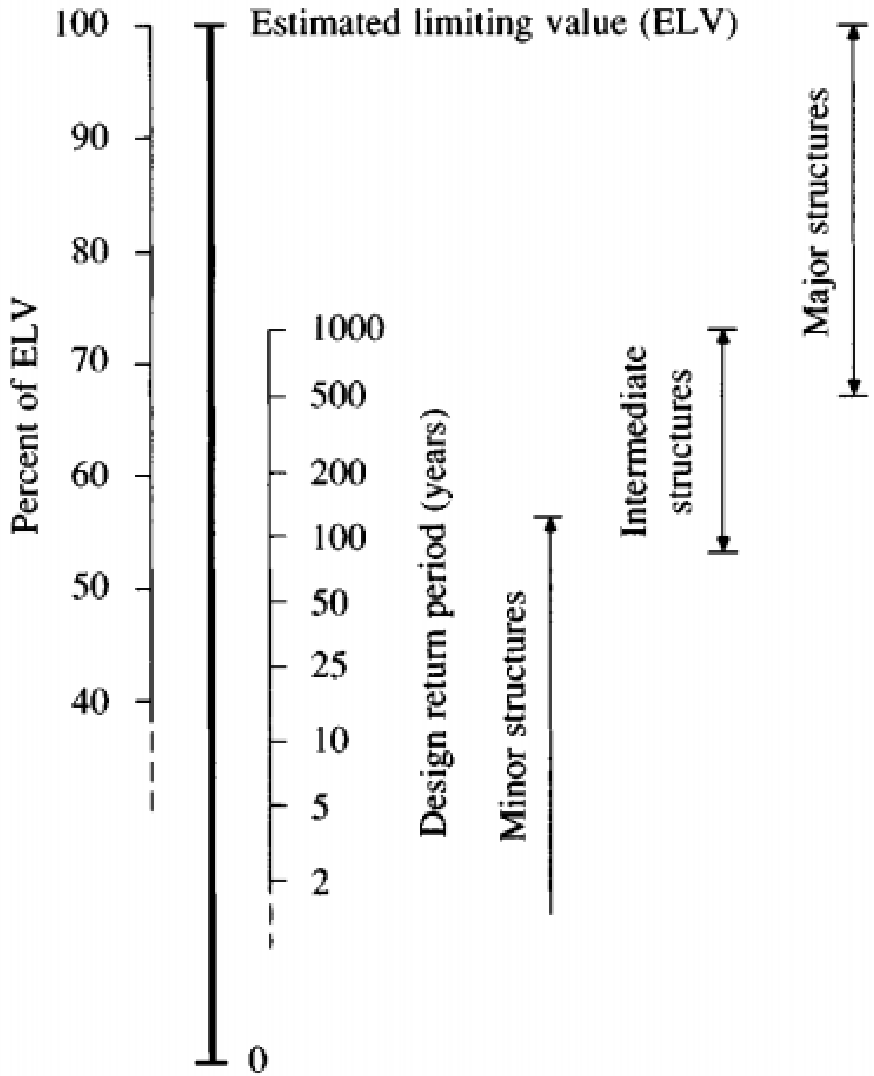



**FIGURE 13.1.1**
Hydrologic design scale. Approximate ranges of the design level for different types of structures are shown. Design may be based on a percentage of the ELV or on a design return period. The values for the two scales shown in the diagram are illustrative only and do not correspond directly with one another.

**TABLE 13.1.1**
**Generalized design criteria for water-control structures**

| Type of structure | Return period (years) | ELV |
|---|---|---|
| Highway culverts | | |
| Low traffic | 5–10 | — |
| Intermediate traffic | 10–25 | — |
| High traffic | 50–100 | — |
| Highway bridges | | |
| Secondary system | 10–50 | — |
| Primary system | 50–100 | — |
| Farm drainage | | |
| Culverts | 5–50 | — |
| Ditches | 5–50 | — |
| Urban drainage | | |
| Storm sewers in small cities | 2–25 | — |
| Storm sewers in large cities | 25–50 | — |
| Airfields | | |
| Low traffic | 5–10 | — |
| Intermediate traffic | 10–25 | — |
| High traffic | 50–100 | — |
| Levees | | |
| On farms | 2–50 | — |
| Around cities | 50–200 | — |
| Dams with no likelihood of loss of life (low hazard) | | |
| Small dams | 50–100 | — |
| Intermediate dams | 100+ | — |
| Large dams | — | 50–100% |
| Dams with probable loss of life (significant hazard) | | |
| Small dams | 100+ | 50% |
| Intermediate dams | — | 50–100% |
| Large dams | — | 100% |
| Dams with high likelihood of considerable loss of life (high hazard) | | |
| Small dams | — | 50–100% |
| Intermediate dams | — | 100% |
| Large dams | — | 100% |

A ***hydrologic design level*** on the design scale is the magnitude of the hydrologic event to be considered for the design of a structure or project.

**Probability of exceedence P**

**Return period T**

**Risk of failure $\bar{R}$**

**Life of the structure n**

## Risk Analysis

Water-control design involves consideration of risks. A water-control structure might fail if the magnitude for the design return period $T$ is exceeded within the expected life of the structure. This *natural*, or *inherent*, hydrologic risk of failure can be calculated using Eq. (12.1.4):

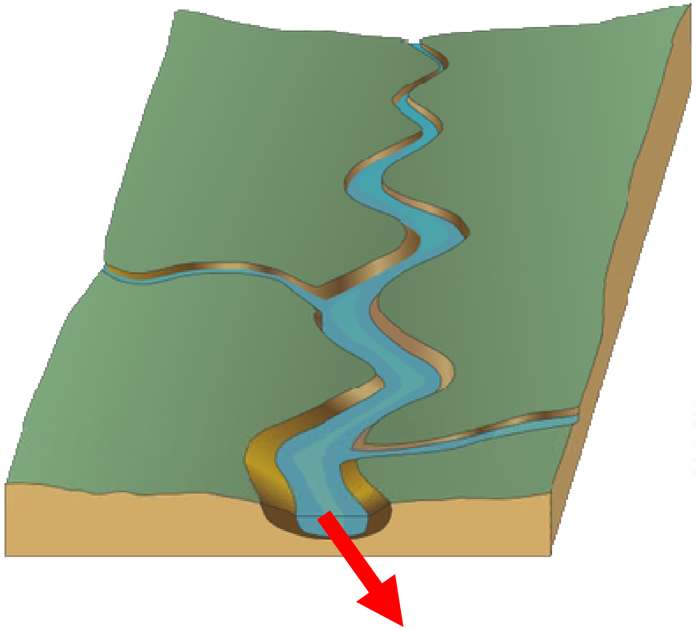
$$\overline{R} = 1 - [1 - P(X \geq x_T)]^n \qquad (13.2.3)$$

where $P(X \geq x_T) = 1/T$, and $n$ is the expected life of the structure; $\overline{R}$ represents the probability that an event $x \geq x_T$ will occur at least once in $n$ years.

## Esempio: rischio idraulico di fallanza di un tombino nella sua vita utile

Design rainfall (x)

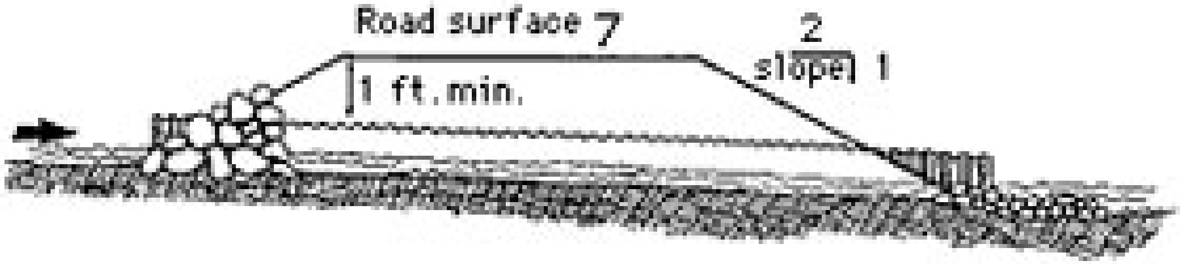
Rainfall runoff model x → Q(x)

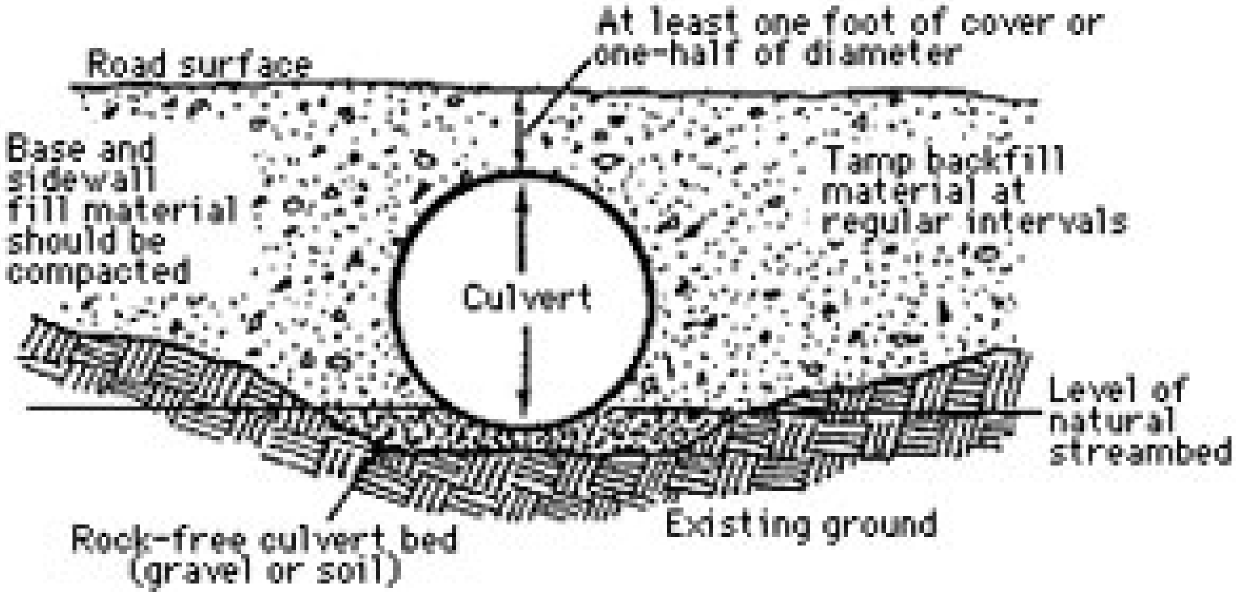


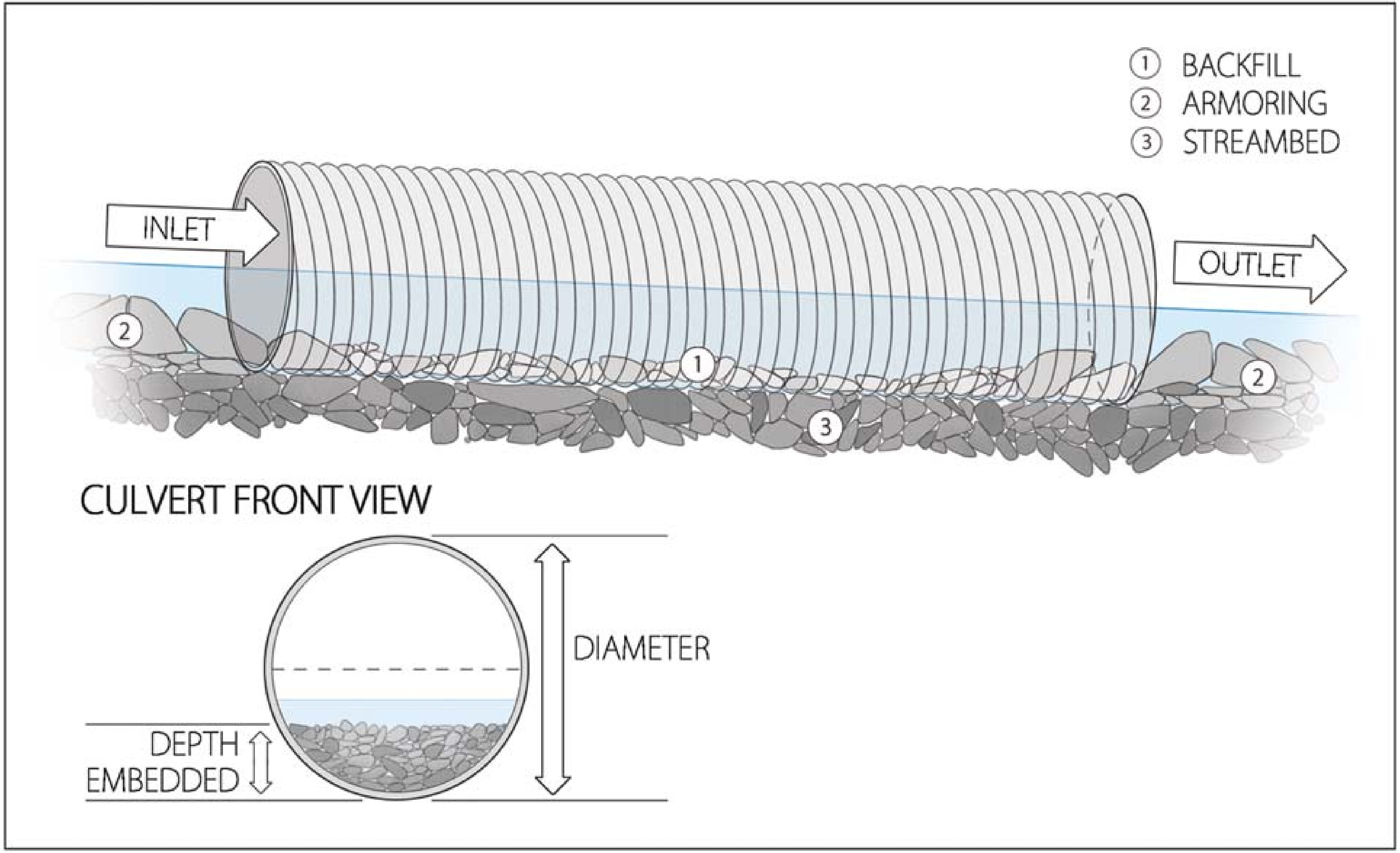
Q(x)

Installation of culvert

(realizzazione di un tombino)

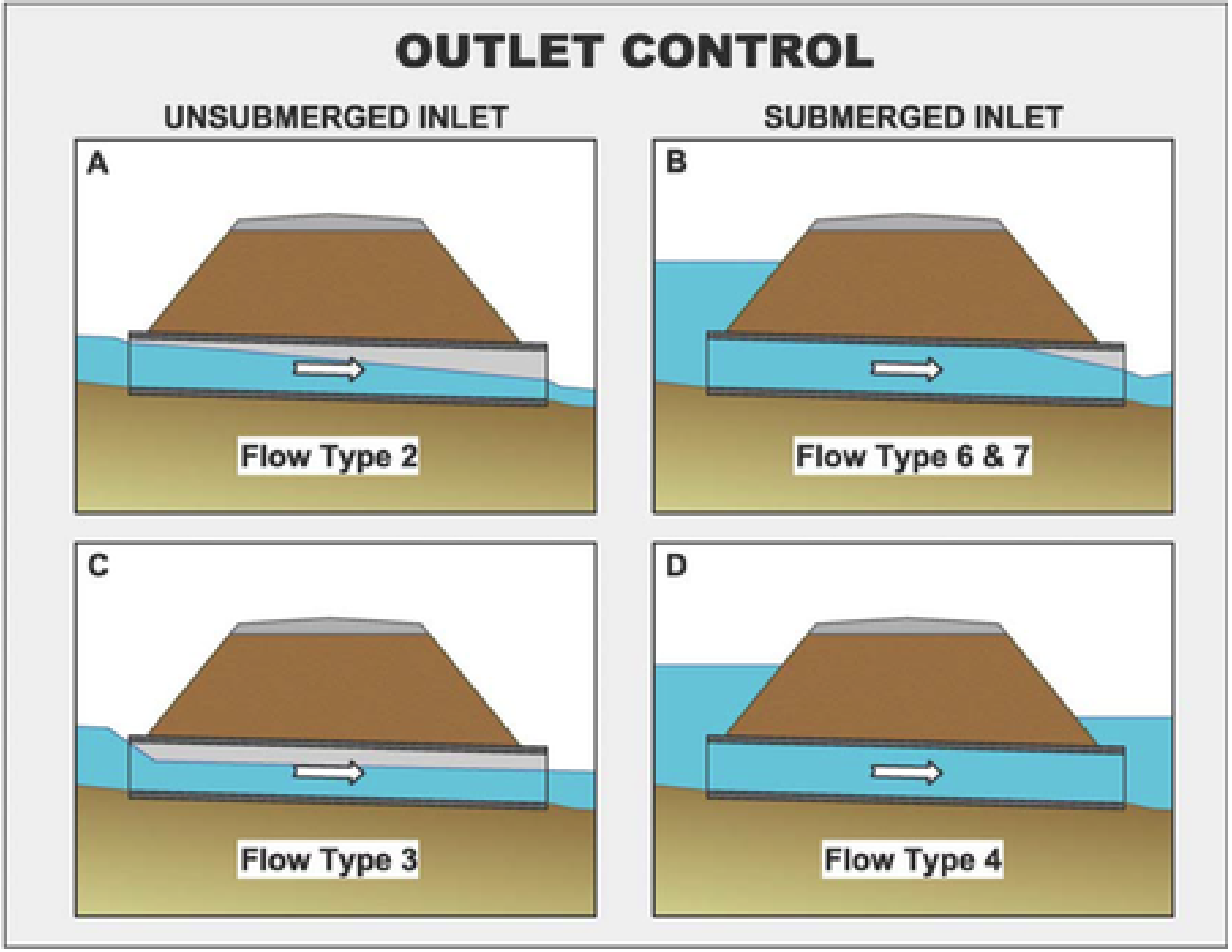


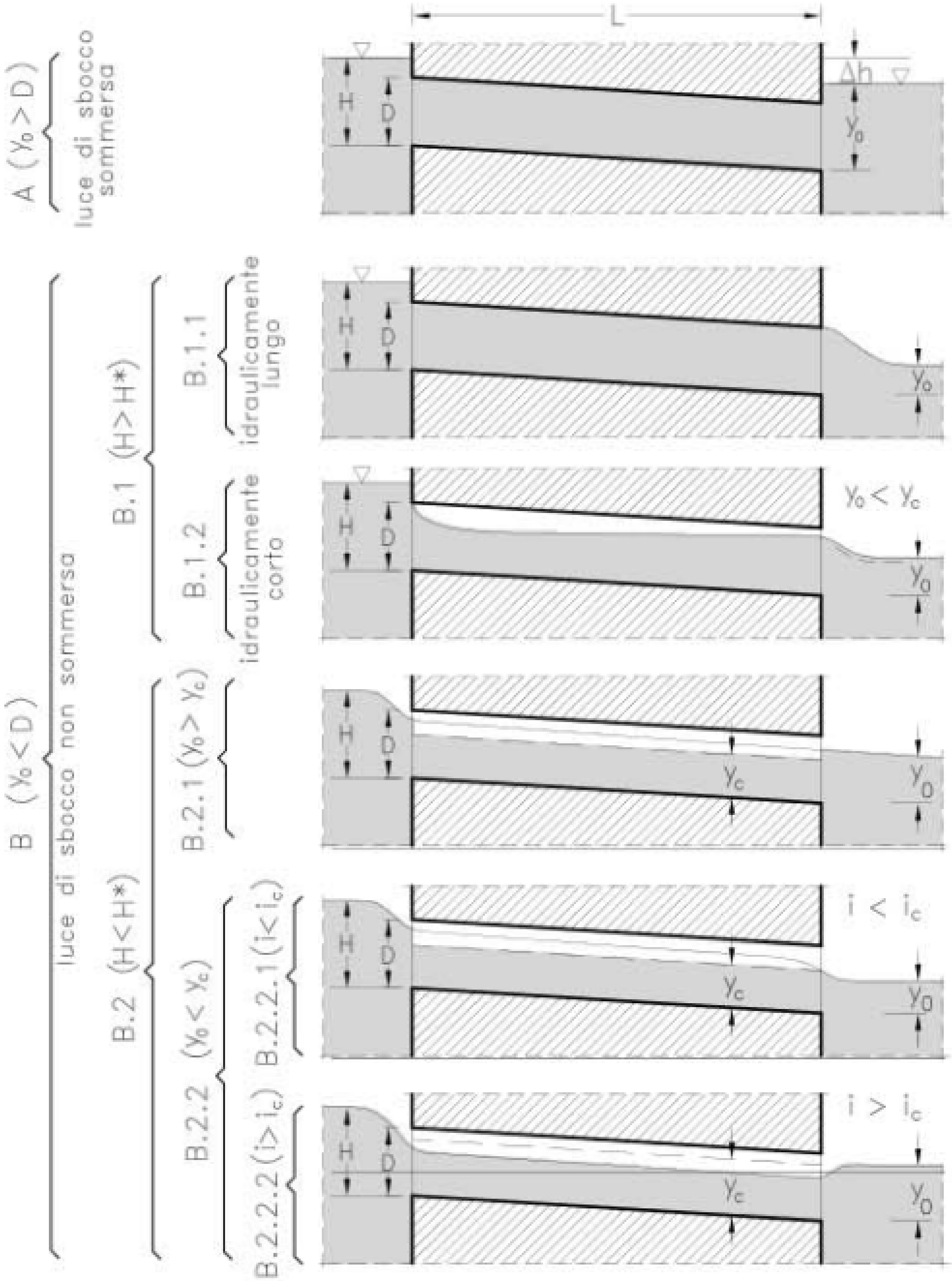





1 BACKFILL
2 ARMORING
3 STREAMBED
INLET
OUTLET
2
1
2
3
CULVERT FRONT VIEW
DIAMETER
DEPTH
EMBEDDED

**Flow model**
**Flow Q(x) ←→ water depth y(x)**



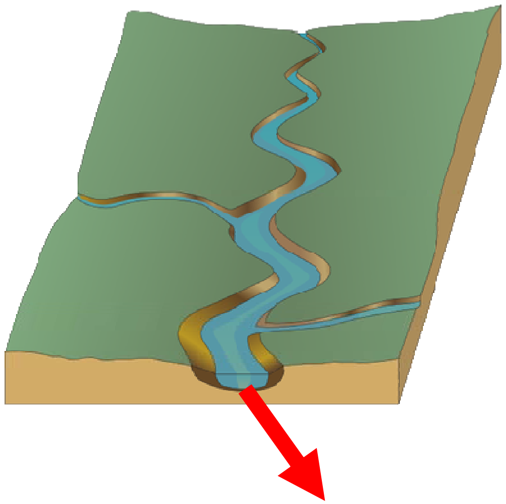



Da Deppo L. e Datei C., “Le opere idrauliche nelle costruzioni stradali”

Cap. 6

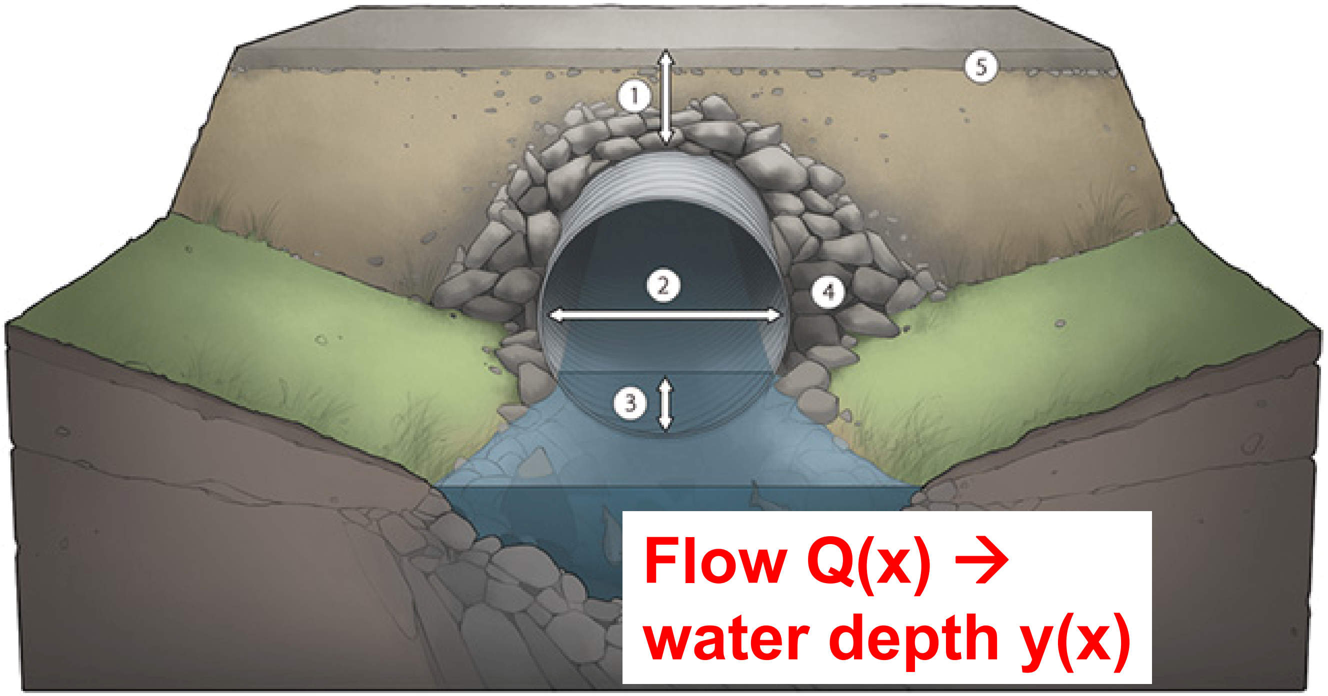
## Design rainfall (x)

## Rainfall runoff model x → Q(x)



Q(x)

**Culvert**




**Flow Q(x) → water depth y(x)**

1. MINIMUM FILL OVER TOP OF CULVERT
2. MINIMUM CULVERT DIAMETER
3. MINIMUM WATER DEPTH
4. ARMORING/REINFORCING AROUND CULVERT
5. ROAD BED

**Example 13.2.2.** A culvert has an expected life of 10 years. If the acceptable risk of at least one event exceeding the culvert capacity during the design life is 10 percent, what design return period should be used? What is the chance that a culvert designed for an event of this return period will not have its capacity exceeded for 50 years?

***Solution.*** By Eq. (13.2.3)

$$\bar{R} = 1 - \left(1 - \frac{1}{T}\right)^n$$

or

$$0.10 = 1 - \left(1 - \frac{1}{T}\right)^{10}$$

and solving yields $T$ = 95 years.

## Elementi rilevanti nell’analisi di rischio che sono stati trascurati

## Safety Factor SF
## (fattore di sicurezza, franco idraulico)

Although natural hydrologic uncertainty can be accounted for as above, other kinds of uncertainty are difficult to calculate. These are often treated using a *safety factor*, SF, or a *safety margin*, SM. Letting the hydrologic design value be $L$ and the actual capacity adopted for the project be $C$, the factor of safety is

$$\mathrm{SF} = \frac{C}{L} \tag{13.2.4}$$

and the safety margin is

$$\mathrm{SM} = C - L \tag{13.2.5}$$

The actual capacity is larger than the hydrologic design value because it has to allow for other kinds of uncertainty: technological (hydraulic, structural, construction, operation, etc.), socioeconomic, political, and environmental.

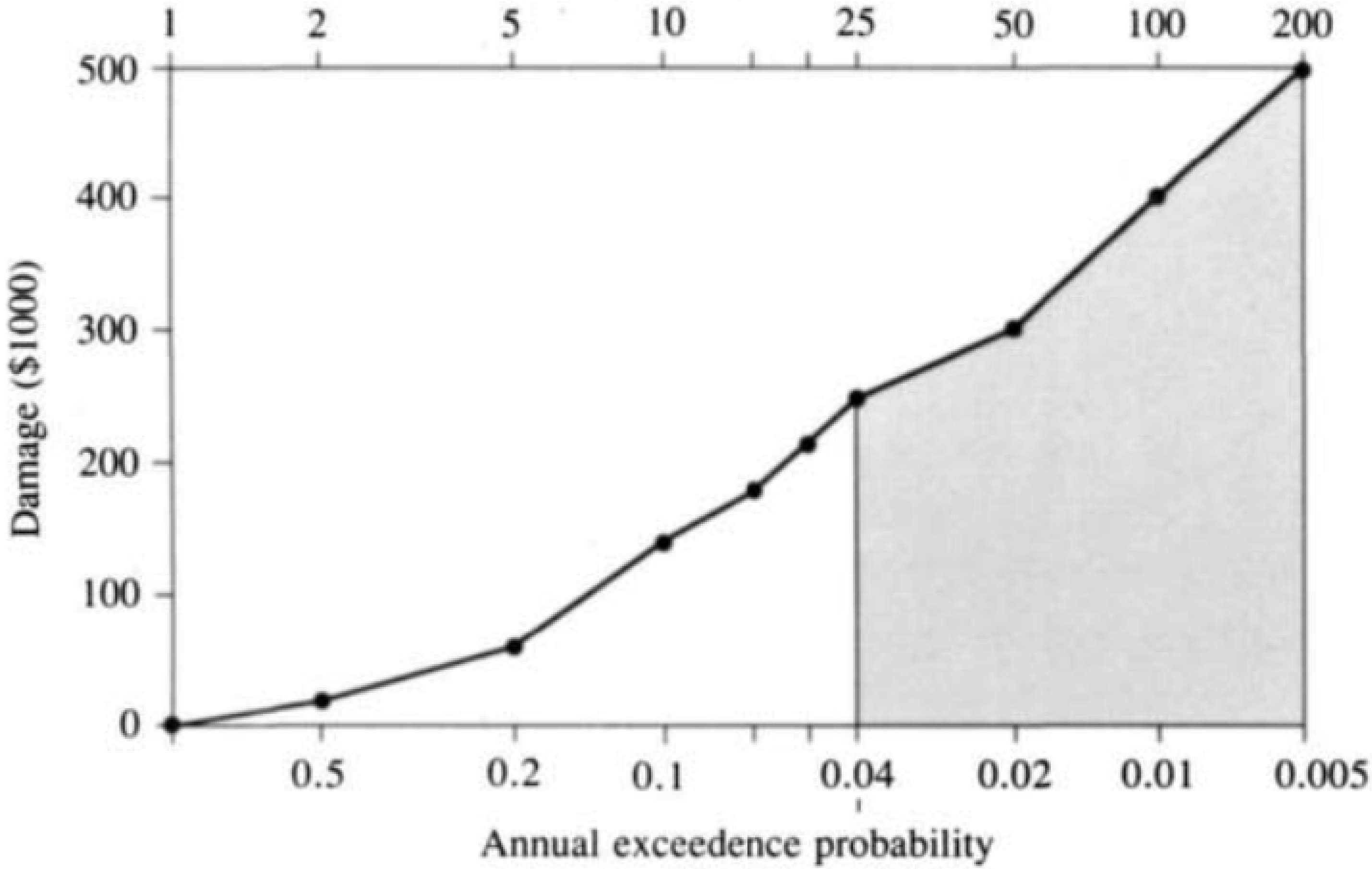
## Hydroeconomic Analysis

The optimum design return period can be determined by hydroeconomic analysis if the probabilistic nature of a hydrologic event and the damage that will result if it occurs are both known over the feasible range of hydrologic events. As the design return period increases, the capital cost of a structure increases, but the expected damages decrease because of the better protection afforded. By summing the capital cost and the expected damage cost on an annual basis, a design return period having minimum total cost can be found.

integrating for $x > x_T$ (the design level). That is, the expected annual cost $D_T$ is

$$D_T = \int_{x_T}^{\infty} D(x) f(x)\, dx \tag{13.2.6}$$

which is the shaded area in Fig. 13.2.2($a$).




($a$) Damages for events of various return periods.

which is approximated by

$$\Delta D_i = \left[\frac{D(x_{i-1}) + D(x_i)}{2}\right]\int_{x_{i-1}}^{x_i} f(x)\,dx$$
$$= \frac{D(x_{i-1}) + D(x_i)}{2}\left[P(x \leq x_i) - P(x \leq x_{i-1})\right] \tag{13.2.8}$$

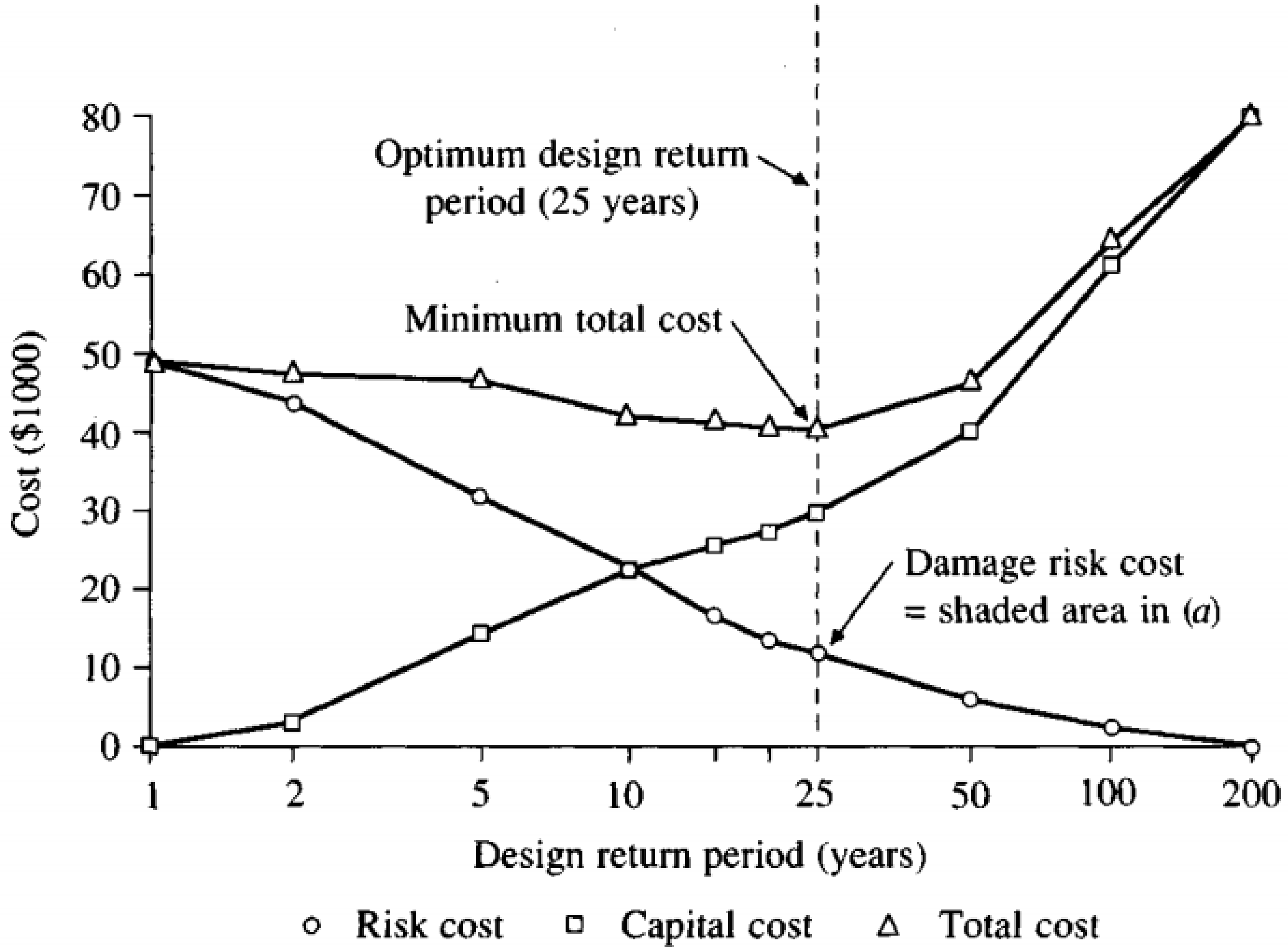
But $P(x \leq x_i) - P(x \leq x_{i-1}) = [1 - P(x \geq x_i)] - [1 - P(x \geq x_{i-1})] = P(x \geq x_{i-1}) - P(x \geq x_i)$, so (13.2.8) can be written

$$\Delta D_i = \frac{D(x_{i-1}) + D(x_i)}{2}\left[P(x \geq x_{i-1}) - P(x \geq x_i)\right] \tag{13.2.9}$$

and the annual expected damage cost for a structure designed for return period $T$ is given by

$$D_T = \sum_{i=1}^{\infty}\left[\frac{D(x_{i-1}) + D(x_i)}{2}\right]\left[P(x \geq x_{i-1}) - P(x \geq x_i)\right] \tag{13.2.10}$$

By adding $D_T$ to the annualized capital cost of the structure, the total cost can be found; the optimum design return period is the one having the minimum total cost.




(*b*) Hydroeconomic analysis.

**FIGURE 13.2.2**
Determination of the optimum design return period by hydroeconomic analysis (Example 13.2.3).

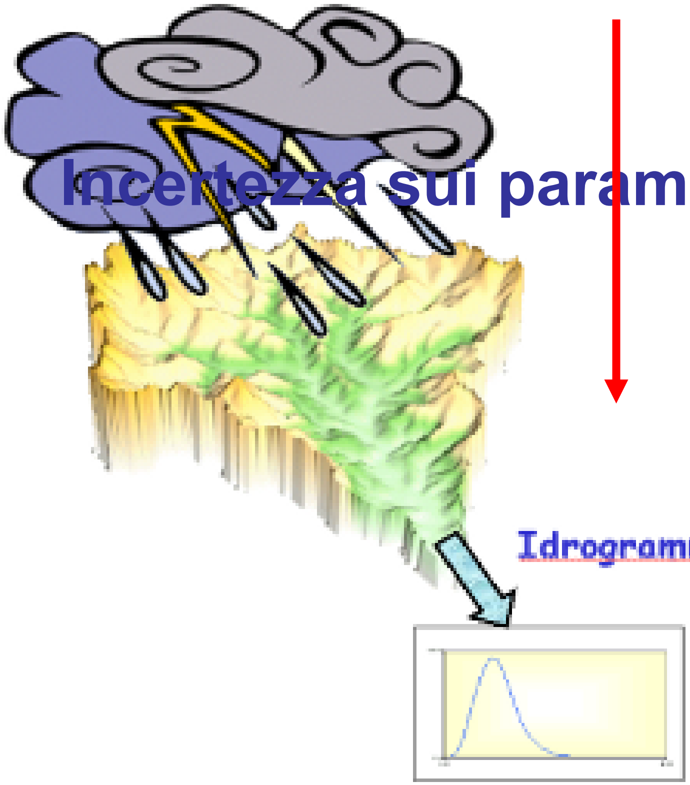
# Fattori che influenzano l'analisi di rischio

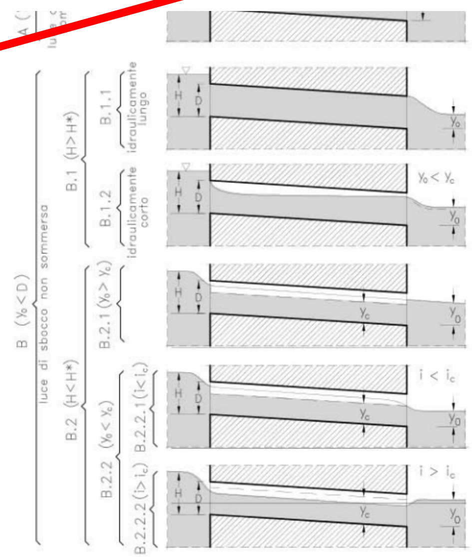
## Incertezza sulle forzanti idrologiche

## Incertezza sul modello

$$\Delta h = k_i \frac{v^2}{2g} + \frac{2gL}{k_s^2 R_H^{4/3}} \cdot \frac{v^2}{2g} + k_s \frac{v^2}{2g}$$

## Incertezza sui parametri del modello



## 13.3 FIRST ORDER ANALYSIS OF UNCERTAINTY

Many of the uncertainties associated with hydrologic systems are not quantifiable. For example, the conveyance capacity of a culvert with an unobstructed entrance can be calculated within a small margin of error, but during a flood, debris may become lodged around the entrance to the culvert, reducing its conveyance capacity by an amount that cannot be predetermined. Hydrologic uncertainty may be broken down into three categories: *natural*, or *inherent*, *uncertainty*, which arises from the random variability of hydrologic phenomena; *model uncertainty*, which results from the approximations made when representing phenomena by equations; and *parameter uncertainty*, which stems from the unknown nature of the coefficients in the equations, such as the bed roughness in Manning's equation. Inherent uncertainty in the magnitude of the design event is described by Eq. (13.2.3); in this section, model and parameter uncertainty will be considered.

The *first order analysis of uncertainty* is a procedure for quantifying the expected variability of a dependent variable calculated as a function of one or more independent variables (Ang and Tang, 1975; Kapur and Lamberson, 1977; Ang and Tang, 1984; Yen, 1986). Suppose $w$ is expressed as a function of $x$:

$$w = f(x) \tag{13.3.1}$$

If the true value of $x$ differs from $\bar{x}$, the effect of this discrepancy on $w$ can be estimated by expanding $f(x)$ as a Taylor series around $x = \bar{x}$:

$$w = f(\bar{x}) + \frac{df}{dx}(x - \bar{x}) + \frac{1}{2!}\frac{d^2f}{dx^2}(x - \bar{x})^2 + \ldots \qquad (13.3.3)$$

where the derivatives $df/dx$, $d^2f/dx^2$, . . . , are evaluated at $x = \bar{x}$. If second and higher order terms are neglected, the resulting *first order* expression for the error in $w$ is

$$w - \bar{w} = \frac{df}{dx}(x - \bar{x}) \qquad (13.3.4)$$

The variance of this error is $s_w^2 = E[(w - \overline{w})^2]$ where $E$ is the expectation operator

$$s_w^2 = E\left\{\left[\frac{df}{dx}(x - \overline{x})\right]^2\right\}$$

or

$$s_w^2 = \left(\frac{df}{dx}\right)^2 s_x^2 \tag{13.3.5}$$

where $s_x^2$ is the variance of $x$.

If $w$ is dependent on several *mutually independent* variables $x_1, x_2, \ldots, x_n$, it can be shown by a procedure similar to the above that

$$s_w^2 = \left(\frac{\partial f}{\partial x_1}\right)^2 s_{x_1}^2 + \left(\frac{\partial f}{\partial x_2}\right)^2 s_{x_2}^2 + \ldots + \left(\frac{\partial f}{\partial x_n}\right)^2 s_{x_n}^2 \qquad (13.3.6)$$

Kapur and Lamberson (1977) show how to extend (13.3.6) to account for the effect on $s_w^2$ of correlation between $x_1, x_2, \ldots, x_n$, if any exists.

## First-Order Analysis of Manning's Equation: Depth as the Dependent Variable

## First-Order Analysis of Manning's Equation: Discharge as the Dependent Variable

## Fattori che influenzano l’analisi di rischio

**Incertezza sulle forzanti idrologiche**

**Incertezza sul modello**

**Incertezza sui parametri del modello**

## Fattori che influenzano l'analisi di rischio

**Incertezza sulle forzanti idrologiche**

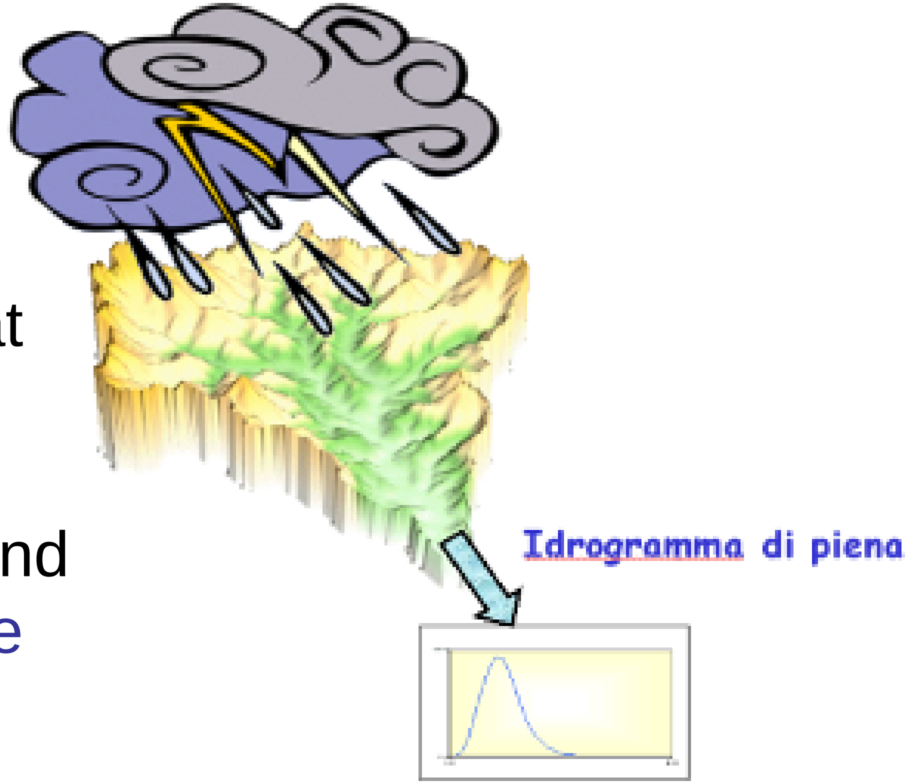
**Incertezza sul modello**

**Incertezza sui parametri del modello**

## Rational method

$$Q=\varphi hA/T_c$$

if a rainfall of intensity i begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration Tc, when all of the watershed is contributing to flow at the outlet.




The product of rainfall intensity i and watershed area A is the inflow rate for the system, iA, and the ratio of this rate to the rate of peak discharge *Q* (which occurs at time *tc)* is termed the runoff coefficient φ

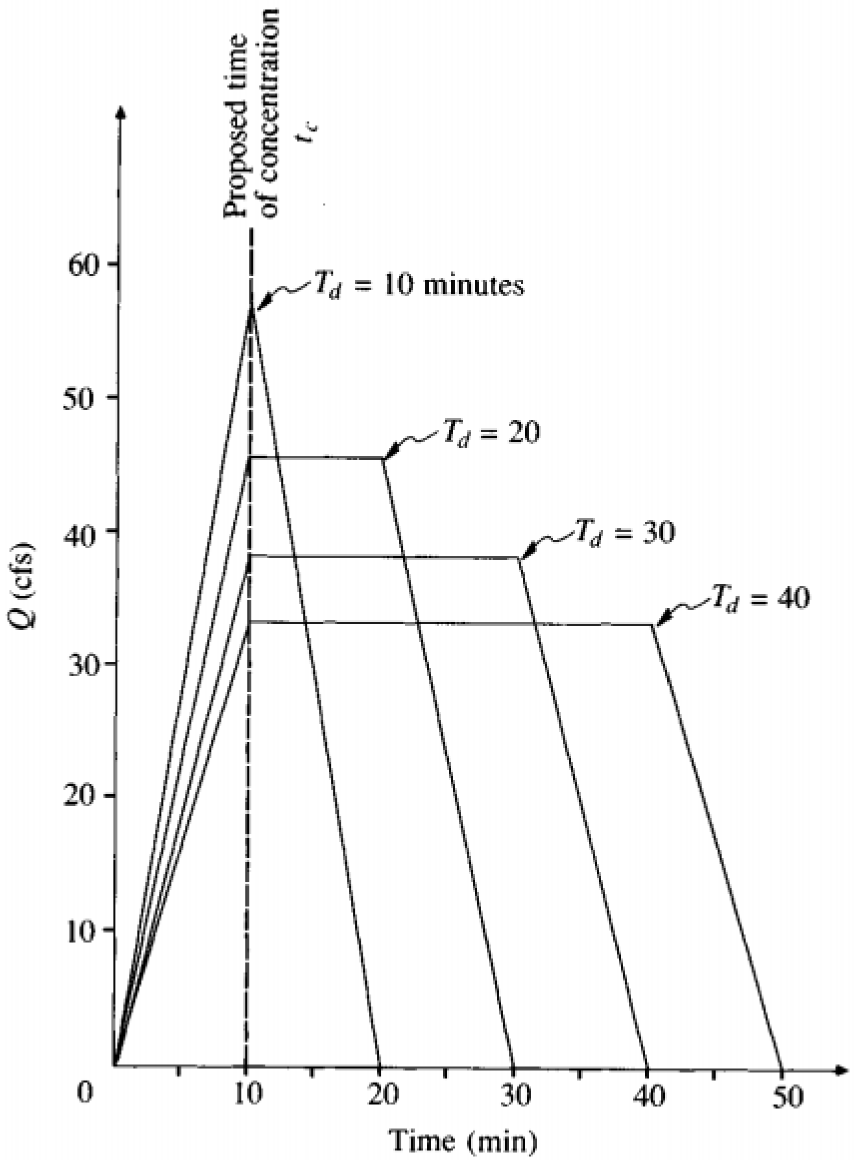
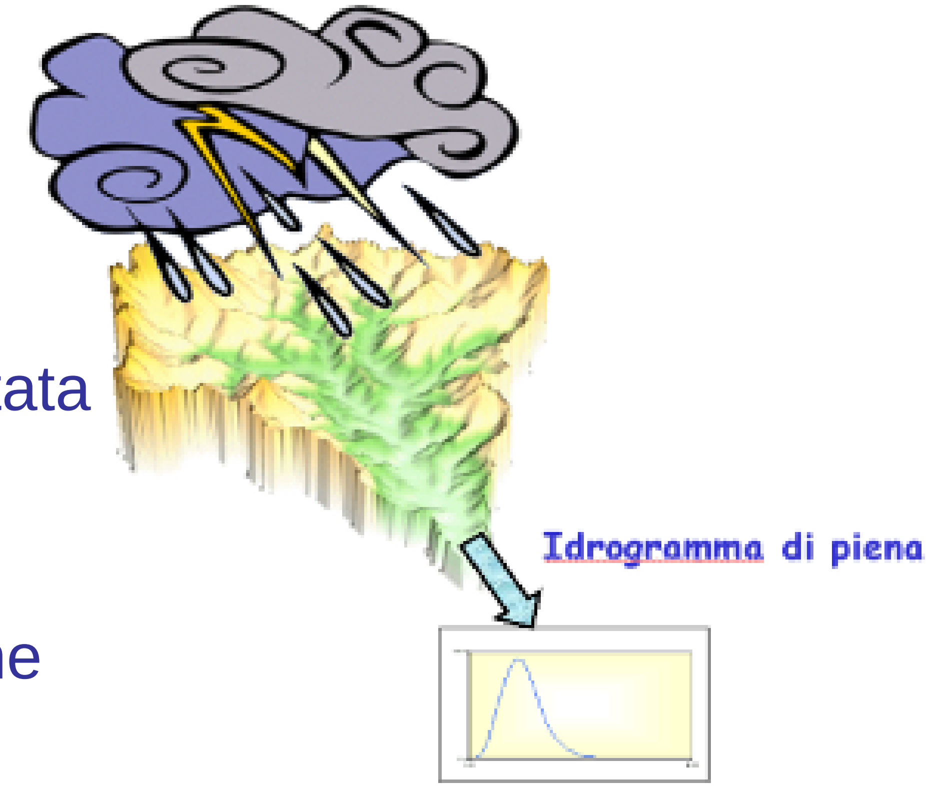



**FIGURE 15.4.3**
Typical storm water runoff hydrographs for the modified rational method with various rainfall durations.

**Incertezza sui parametri del modello**

**Incertezza sulle forzanti idrologiche**

$$Q=\varphi \; h \; A/Tc$$



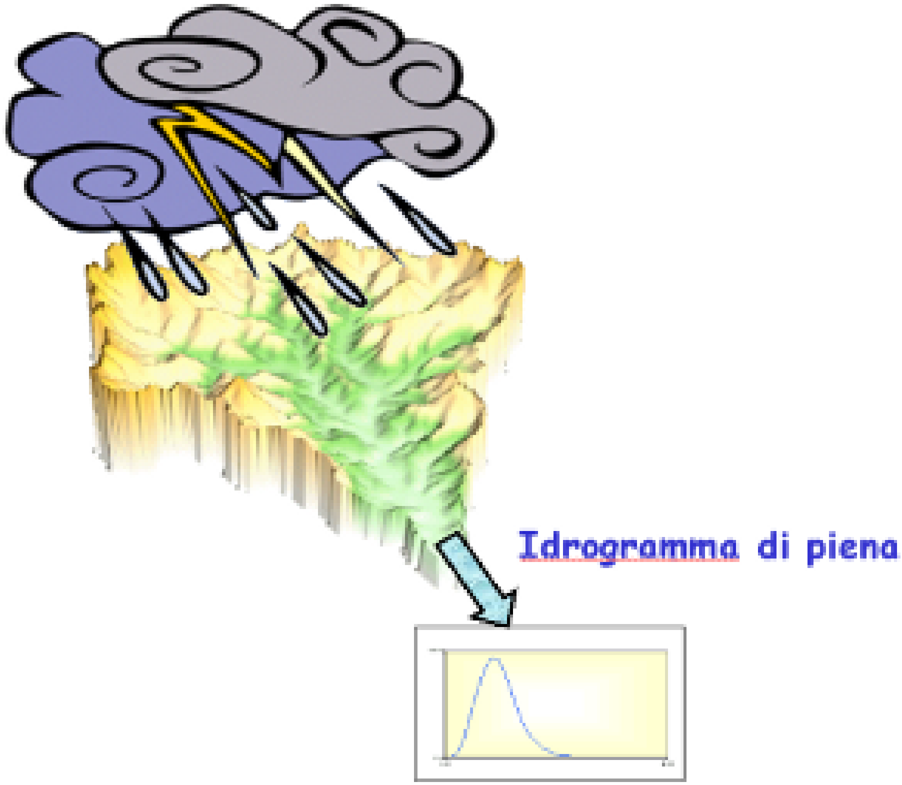

Esempio: calcolo della portata di piena in un bacino, assumendo

1) Che solo la precipitazione sia aleatoria

2) Che sia la precipitazione che il coefficiente di deflusso siano aleatori

## Dati

| | | |
|---|---|---|
| S | 100 | km2 |
| | 100000000 | m2 |
| Tc | 12 | ore |
| | 43200 | s |
| $\varphi$ | 0,40 | |




## Momenti delle precipitazioni massime annuali di durata Tc

| | | |
|---|---|---|
| Media: $h_{med}$ | 50,4 | mm |
| Scarto quadratico medio: $s_h$ | 21,9 | mm |

## Distribuzione di Gumbel (h)

$$P(h) = exp\left[-exp-\left(\frac{h-a}{b}\right)\right]$$

$$h_{med} = a + b \cdot 0{,}577$$

$$s_h = \frac{\pi}{\sqrt{6}} b$$

| | | |
|---|---|---|
| b(Gumbel) | 17,084021 | mm |
| a(Gumbel) | 40,54252 | mm |
| | | |
| Tr | 100 | |
| P | 0,99 | |
| y | 4,6001492 | |
| h | 119,13157 | mm |
| | 0,1191316 | m |
| Q | 110,31 | m3/s |

$$P = 1 - \frac{1}{Tr}$$

$$y = -ln\left[-ln\left(1 - \frac{1}{Tr}\right)\right]$$

$$y = \frac{h-a}{b} \Rightarrow h = by + a$$

**Q=φ h A/Tc**

## Distribuzione di Gumbel (Q)

$$P(Q(h)) = exp\left[-exp-\left(\frac{QTc - a\varphi S}{\varphi Sb}\right)\right] = exp\left[-exp-\left(\frac{Q - a_Q}{b_Q}\right)\right]$$

Q=φ h A/Tc

| Gumbel Q | |
|---|---|
| b(Q) | 15,82 |
| a(Q) | 37,54 |
| Qmed | 46,67 |
| sQ | 20,277778 |

$$Q_{med} = a_Q + b_Q \cdot 0{,}577$$

$$s_Q = \frac{\pi}{\sqrt{6}} b_Q$$

# Analisi del primo ordine → $s_Q$

If the true value of $x$ differs from $\bar{x}$, the effect of this discrepancy on $w$ can be estimated by expanding $f(x)$ as a Taylor series around $x = \bar{x}$:

$$w = f(\bar{x}) + \frac{df}{dx}(x - \bar{x}) + \frac{1}{2!}\frac{d^2f}{dx^2}(x - \bar{x})^2 + \ldots \qquad (13.3.3)$$

where the derivatives $df/dx$, $d^2f/dx^2$, . . . , are evaluated at $x = \bar{x}$. If second and higher order terms are neglected, the resulting *first order* expression for the error in $w$ is

$$w - \bar{w} = \frac{df}{dx}(x - \bar{x}) \qquad (13.3.4)$$

# Analisi del primo ordine → $s_Q$

The variance of this error is $s_w^2 = E[(w - \bar{w})^2]$ where $E$ is the expectation operator

$$s_w^2 = E\left\{\left[\frac{df}{dx}(x - \bar{x})\right]^2\right\}$$

or

$$s_w^2 = \left(\frac{df}{dx}\right)^2 s_x^2 \qquad (13.3.5)$$

where $s_x^2$ is the variance of $x$.

## Analisi del primo ordine → $s_Q$

1) Solo la precipitazione sia aleatoria

$$Q_{med} = \frac{\varphi S h_{med}}{Tc}$$

$$s_Q = \sqrt{\left(\frac{\partial Q}{\partial h} s_h\right)^2} = \frac{\partial Q}{\partial h} s_h$$

2) Sia la precipitazione che il coefficiente di deflusso siano aleatori

$$Q_{med} = \frac{\varphi_{med} S h_{med}}{Tc}$$

$$s_Q = \sqrt{\left(\frac{\partial Q}{\partial h} s_h\right)^2 + \left(\frac{\partial Q}{\partial \varphi} s_\varphi\right)^2}$$

# Analisi del primo ordine → $s_Q$

1) Solo la precipitazione sia aleatoria

$$Q_{med} = a_Q + b_Q \cdot 0{,}577$$

$$s_Q = \frac{\pi}{\sqrt{6}} b_Q$$

| | 1) | 2) |
|---|---|---|
| sQ | 20,28 | 50,8 |
| bQ | 15,82 | 39,69 |
| aQ | 37,54 | 23,76 |
| yQ | 4,60 | 4,60 |
| Q(Tr) | 110,3 | 206,4 |

2) Sia la precipitazione che il coefficiente di deflusso siano aleatori

$$P(Q(Tr)) = 1 - \frac{1}{Tr} = exp\left[ - exp - \left( \frac{Q - a_Q}{b_Q} \right) \right]$$


Ridotta capacità di portata del tombino→
ridotta capacità di far fronte al carico Q(x)

**Analisi di rischio in caso di carico e resistenza aleatori**

## 13.4 COMPOSITE RISK ANALYSIS

The previous sections have introduced the concepts of inherent uncertainty due to the natural variability of hydrologic phenomena, and model and parameter uncertainty arising from the way the phenomena are analyzed. Composite risk analysis is a method of accounting for the risks resulting from the various sources of uncertainty to produce an overall risk assessment for a particular design. The concepts of loading and capacity are central to this analysis.

The *loading*, or *demand*, placed on a system is the measure of the impact of external events. The demand for water supply is determined by the people who use the water. The magnitude of a flash flood depends on the characteristics of the storm producing it and on the condition of the watershed at the time of the storm. The *capacity*, or *resistance*, is the measure of the ability of the system to withstand the loading or meet the demand.

If loading is denoted by $L$ and capacity by $C$, then the risk of failure $\bar{R}$ is given by the probability that $L$ exceeds $C$, or

$$\begin{aligned} \bar{R} &= P\left(\frac{C}{L} < 1\right) \\ &= P(C - L < 0) \end{aligned} \qquad (13.4.1)$$

The risk depends upon the probability distributions of $L$ and $C$. Suppose that the probability density function of $L$ is $f(L)$.

The true capacity is not known exactly, but may be considered to have probability density function $g(C)$, which could be the normal or lognormal distribution arising from the first-order analysis of uncertainty in the system capacity.

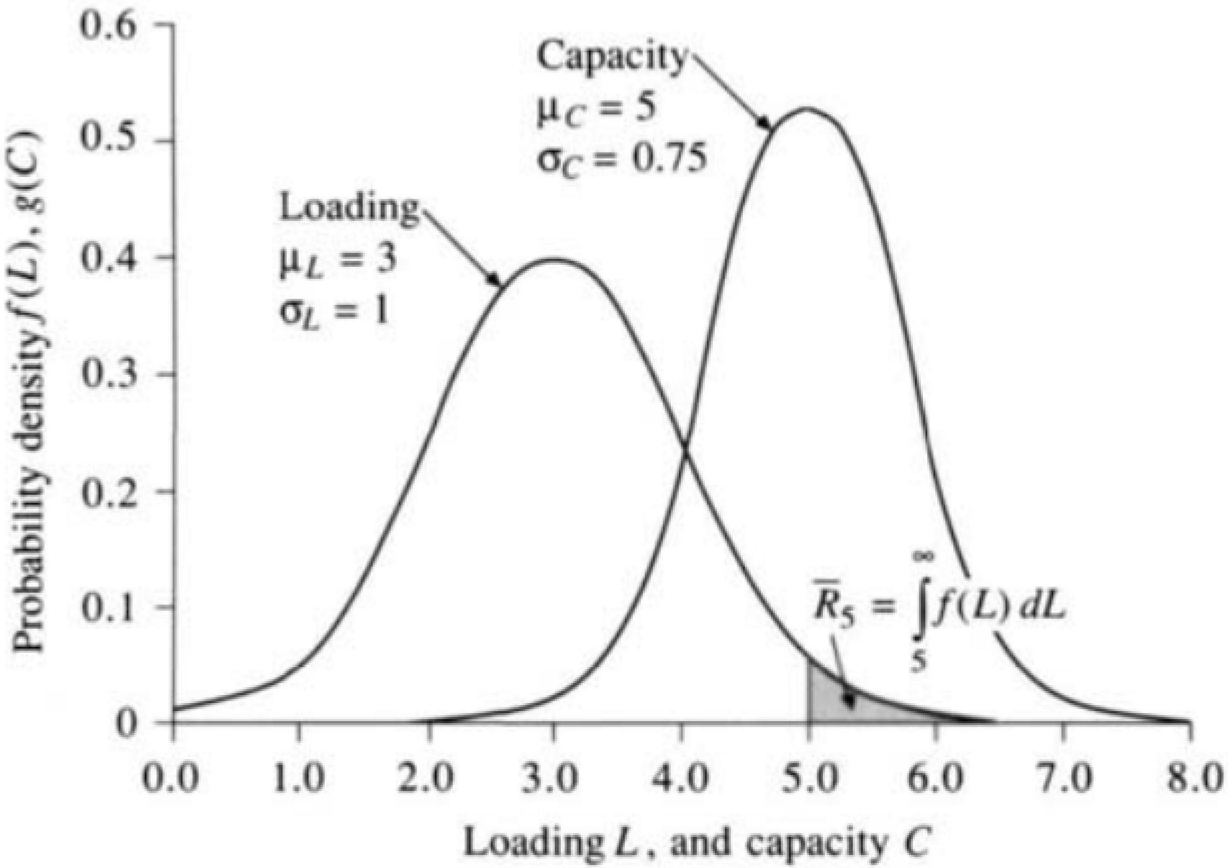



**FIGURE 13.4.1**
Composite risk analysis. Area shaded is the risk $\overline{R}_5$ of the loading exceeding a fixed capacity of 5 units. The risk that the loading will exceed the capacity when the capacity is random is given by $\overline{R} = \int_{-\infty}^{\infty} [\int_C^\infty f(L)\,dL]\, g(C)\,dC$. The loading and capacity shown are both normally distributed (Example 13.4.1).

$$\overline{R} = \int_{-\infty}^{\infty} \left[ \int_C^{\infty} f(L)\,dL \right] g(C)\,dC$$

The *reliability* of a system is defined to be *the probability that a system will perform its required function for a specified period of time under stated conditions* (Harr, 1987). Reliability $R$ is the complement of risk, or the probability that the loading will not exceed the capacity:

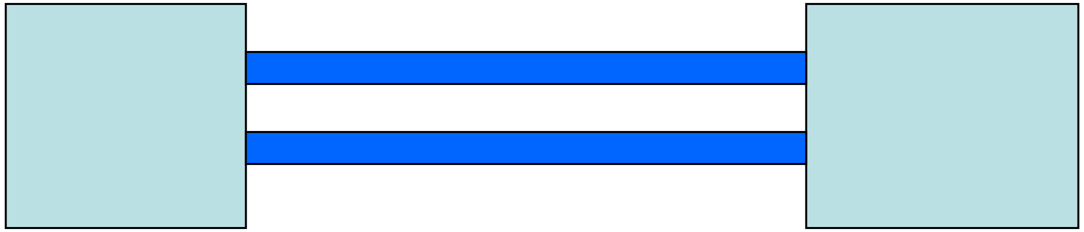
$$R = P(L \leq C) \qquad (13.4.4)$$
$$= 1 - \overline{R}$$

or

$$R = \int_{-\infty}^{\infty} \left[ \int_{0}^{C} f(L)\, dL \right] g(C)\, dC \qquad (13.4.5)$$

2 elementi in parallelo

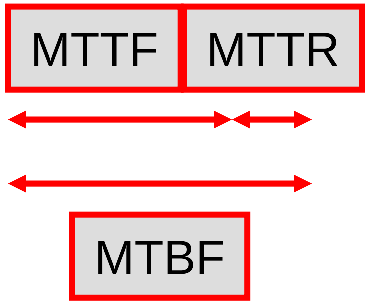


ostruiti 4 volte/anno

riabilitati in 20 gg

MTTF 365/4=91,25 gg

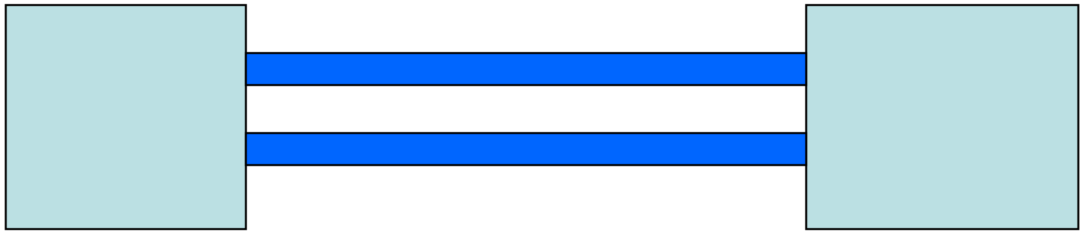
MTTR 20 gg




$$a = affidabilità = \frac{MTTF}{MTTF + MTTR} = \frac{\frac{1}{\lambda}}{\frac{1}{\mu} + \frac{1}{\lambda}} = \frac{\mu}{\mu + \lambda}$$

$$1 - a = \frac{\lambda}{\mu + \lambda}$$

2 elementi in parallelo

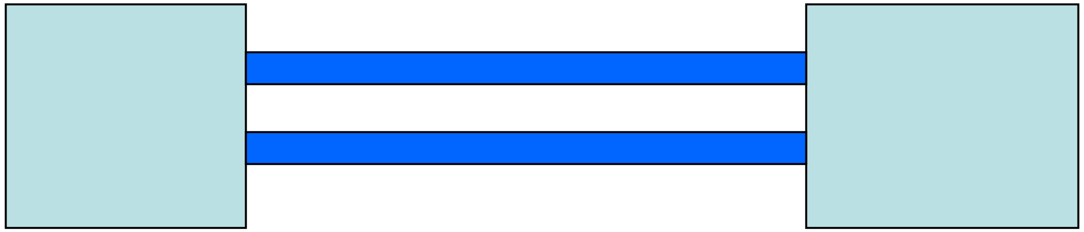


| | |
|---|---|
| ostruiti 4 volte/anno | |
| riabilitati in 20 gg | |
| MTBF | 111,25 |
| MTTR | 20 |
| Affidabilità (1 elemento) | **0,82** |

# Affidabilità del sistema

## Solo la precipitazione sia aleatoria

| | |
|---|---|
| b(Q) | 15,82 |
| a(Q) | 37,54 |



$$P(Q) = exp\left[-exp-\left(\frac{Q-a_Q}{b_Q}\right)\right] = 0{,}99$$

$$P(Q/2) = 0{,}72$$

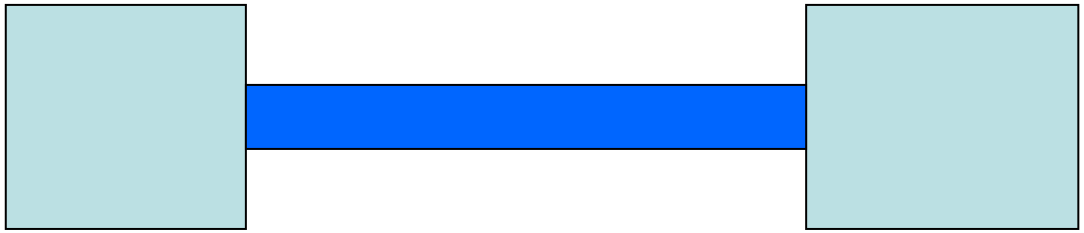
$$A = P(Q/2)\cdot P\left[C = \frac{Q}{2}\right] + P(Q)\cdot P[C = Q]$$

$$P(Q/2)\cdot 2a(1-a) + P(Q)\cdot a^2 \underline{= 0{,}88}$$

# Affidabilità del sistema

## Solo la precipitazione sia aleatoria

| | |
|---|---|
| b(Q) | 15,82 |
| a(Q) | 37,54 |



1 elemento

ostruito 3 volte/anno

riabilitato in 5 gg

| | |
|---|---|
| MTBF | 126,7 |
| MTTR | 5 |
| Affidabilità (1 elemento) | 0,96 |

$$P(Q) = exp\left[-exp-\left(\frac{Q-a_Q}{b_Q}\right)\right] = 0{,}99$$

$$A = P(Q)\cdot P[C=Q] = P(Q)\cdot a = \underline{0{,}95}$$

## 13.5 RISK ANALYSIS OF SAFETY MARGINS AND SAFETY FACTORS

### Safety Margin

The safety margin was defined in Eq. (13.2.5) as the difference between the project capacity and the value calculated for the design loading $SM = C - L$. From (13.4.1), the risk of failure $\bar{R}$ is

$$\begin{aligned} \bar{R} &= P(C - L < 0) \\ &= P(SM < 0) \end{aligned} \tag{13.5.1}$$

If $C$ and $L$ are independent random variables, then the mean value of SM is given by

$$\mu_{SM} = \mu_C - \mu_L \tag{13.5.2}$$

and its variance by

$$\sigma_{SM}^2 = \sigma_C^2 + \sigma_L^2 \tag{13.5.3}$$

so the standard deviation, or standard error of estimate, of the safety margin is

$$\sigma_{SM} = \left(\sigma_C^2 + \sigma_L^2\right)^{1/2} \tag{13.5.4}$$