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# Re-Gen
# Risk Assessment of Ageing Infrastructure

# Risk Optimization in Road Infrastructure Elements

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# Executive summary

This deliverable consists of a report on the risk analysis and risk optimization of road infrastructures including bridges, retaining walls, and slopes as important elements in road transportation networks. In the risk analysis of the above-mentioned infrastructures, the fault tree technique has been employed to investigate the root causes and failure modes while considering deterioration mechanisms (e.g., ageing), climate-change stresses, and traffic growth. These failure modes could result in partial/total collapse or the loss of serviceability/functionality of the infrastructures. As such, a consequence analysis has also been carried out by considering direct and indirect costs such as vehicle operating costs, travel time costs, and accident costs. Having the probability of failures and the corresponding consequences, the risk of each failure mode can be identified as the product of the probability and the total monetary value of direct and indirect financial and operational consequences. Finally, the risks can be optimized by employing a cost-benefit analysis in which based on a comparison between the cost of remedial actions (e.g., preventive maintenance, retrofitting) and the residual risks (benefit).

# 1. Introduction

The majority of infrastructure components for road transport systems in Europe were constructed during the 1960s and the 1970s. Many of the infrastructures built during this period are now in need of repair or can no longer adequately serve the road users due to the aging of the infrastructures on the one hand and the increasing load of traffic on the other hand. In addition, deterioration mechanisms boosted by climate-change stresses (e.g., floods), corrosive environments (e.g., acidic rains), and traffic growth have resulted in a higher frequency of failures which not only endangers the life of road users but also results in higher repair costs and maintenance to restore the required level of performance of road infrastructure. Thus, there is a significant need for risk assessment and risk management in order to assess and optimize the risk based on limited available budgets/resources.

Many researchers have pinpointed the importance of adopting a risk-based optimization/maintenance approach for road infrastructures. Adey et al. (2003) develop a risk-based approach based on limit state equations (load-capacity analysis) to determine optimal interventions for bridges affected by multiple hazards (e.g. traffic load, excessive scour leading to foundation failure). In their work both the structure failures and the inadequate service levels have been considered. Similar work can be found in Decò and Frangopol (2011) and Lacoste et al. (2012).

In the present study, the failure refers to both structural failures (such as those caused by weather, traffic growth, and infrastructure management failures) and functional failures (such as those caused by traffic jams due to extreme weather conditions).. Total collapse refers to a complete failure in permanent state (e.g. a bridge collapse due to extreme scour), while partial failure (also known as critical defect) refers to the conditions in which the infrastructure has undergone some deformation or section loss, but has not yet completely failed. Infrastructure failures can cause interruption to commercial activities and services, resulting in significant repair costs and threatening the safety of human life. Functional failures refer to the cases in which the infrastructure cannot provide defined levels of service in the temporary state. For instance, extreme weather conditions can reduce the capacity of a bridge in the form of lane closure due to heavy floods.

In Section 2 of this report, the hazards and relevant failure modes are identified for bridges, retaining walls, and slopes, and accordingly a fault tree methodology is developed to investigate the root causes of failures. Section 3 considers the consequence analysis of infrastructure failures including both direct and indirect consequences. Defining the risk as the multiplication of the failure probability and the ensuing consequences is described in Sections 4 and in Section 5 a risk-based cost-benefit analysis approach is proposed to minimize the risk. The suggestions and concluding remarks can be found in Section 6.

# 2. Quantitative Risk Assessment

## 2.1. Fault tree analysis

Many approaches have been developed for accident analysis, among which Fault Tree (FT) analysis is the most common technique (Bobbio et al., 2001). FT is a deductive, structured methodology to determine the potential causes of an undesired event such as a system failure or unavailability, referred to as the top event (TE).

While the TE is placed at the top of the tree, the tree is constructed downwards, dissecting the system to intermediate events (IEs in Figure 1) and the primary events (PEs in Figure 1) leading to the TE. PEs are considered binary (with two states) and statistically independent. In an FT, the relationships between events are represented by means of gates, of which AND-gates and OR-gates are the most widely used (Figure 1).

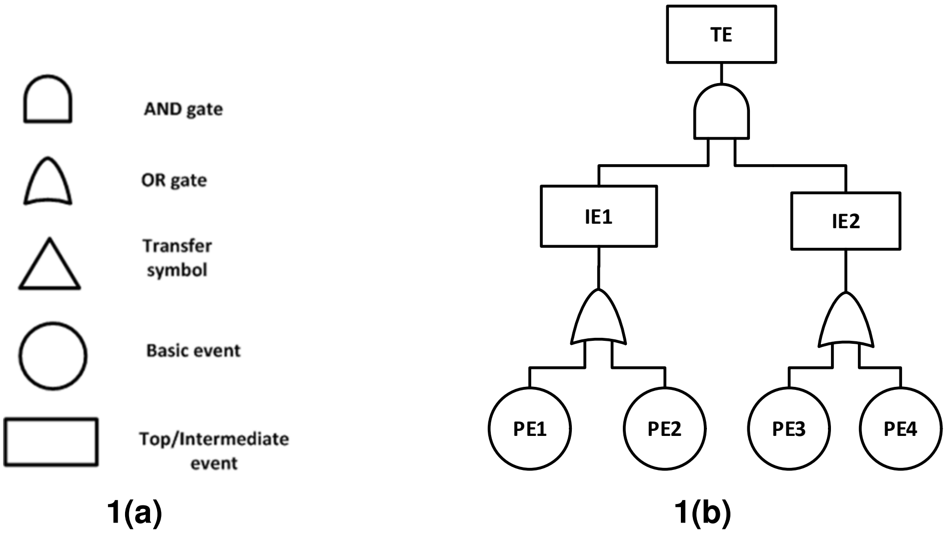


**Figure 1. Schematic representation of fault tree and its typical symbols.**

Once completed, the FT can be analysed both qualitatively and quantitatively using Boolean algebra. In qualitative analysis, an expression is derived for the TE in terms of combinations of PEs while in the quantitative analysis the probability of the TE is calculated based on the occurrence/failure probability of the PEs.

Small FTs can be evaluated manually, however large and complex FTs require the aid of computerized methods for evaluation such as Monte Carlo simulation. Analytical approaches such as minimal cut sets (MCSs) determination are also used for evaluation of FTs. In fault tree analysis, a cut set is a combination of root causes which could lead to the failure of the top event; for example, in the fault tree of Figure 1(b) the failures of the root causes PE1, PE2, and PE3 can result in the failure of the top event. As a result, PE1-PE2-PE3 can be considered as a cut set.

Accordingly, a MCS is a minimum number of root causes whose failures can result in the failure of the top event. For example, in the fault tree of Figure 1(b), PE1-PE3, PE1-PE4, PE2-PE3, and PE2-PE4 are the minimal cut sets of the fault tree. In other words, each MCS can be deemed as a critical mode of failure of the system under consideration. To model the uncertainty arisen from inaccuracy or incompleteness of the data, usually fuzzy set theory and evidence theory are used to handle probabilities in FT analysis (Ferdous et al., 2009; Markowski et al., 2009).

Thus, in the present work we have applied a FT technique to qualitatively and quantitatively (subject to the availability of data) explain possible modes of failure of road infrastructures, down to the level of detail of the PEs.

## 2.2. Causes of failure

### 2.2.1. Bridge failure

In order to better investigate the causes of failures and failure mechanisms we divide the main structure of a bridge to the superstructure and substructure elements (Figure 2). Depending on whether it is a steel bridge or concrete bridge, each of these elements comprises a number of components. The superstructure mainly refers to the spans of a bridge whereas the substructure includes bearings, piers, foundation, and abutments.

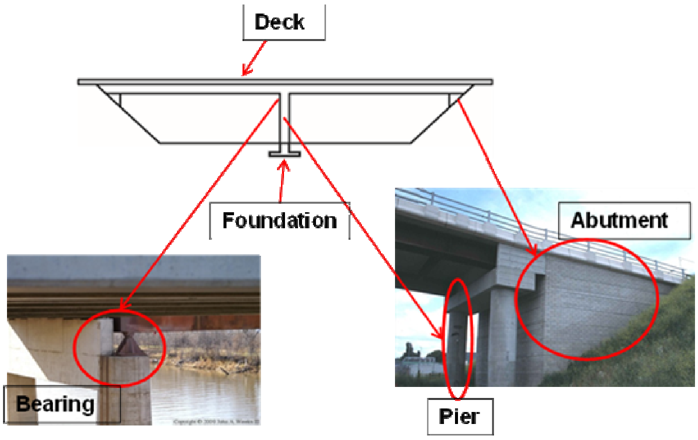


**Figure 2. Main structural elements of a bridge.**

Bridge failures can also be categorized as total collapse (Figure 3a ), partial collapse (Figure 3b), or loss of serviceability. Total collapse is defined as a structural condition where all primary members of a span(s) have undergone severe deformation or damage such that no vehicle passage is possible or safe (due to imminent collapse of the bridge).

Partial collapse is defined as a structural condition where all or some of the primary structural members of a span(s) have undergone some degrees of deformation/damage such that the lives of those travelling on or under the structure would be in danger. Reportedly, the most common bridges to fail are steel construction and beam/girder bridge types (Cook, 2014).

3(a)


3(b)

**Figure 3. Types of structural failures of bridges: total collapse (3a) and partial collapse (3b).**

Based on a literature review (Wardhana and Hadipriono, 2003; Sharma and Mohan, 2011; Cook, 2014; cook et al., 2015), the main causes of failure were determined as hydraulic events (53.48%), collision (14.55%), overload (11%), deterioration (7.23%), fire (3.06%), construction error (2.07%), and fatigue (0.93%). The numbers in the brackets refer to the mean value of the numbers reported in the literature. Further, the mean age at total collapse is 54.8 years, and the median is 51 years (Cook et al., 2015), signifying that half of the bridge failures occur before the bridge reaches the age of 51years.

The hydraulic events include flood, scour, tidal, and debris build-up, with scour being the most prevalent. In particular, flood-induced scour can lead to vertical or horizontal displacement of piers which in turn could cause the collapse of bridge spans. Furthermore, other natural events such as earthquake and variation in the level of ground water can reduce the soil bearing capacity and thus lead to pier displacement.

Collision or lateral impact is the second leading cause of bridge failure. Compared to hydraulic events, collision has rarely led to total collapse. Most of these failures were attributed to lateral impact forces of vehicles on bridges' spans and piers. Examples are collisions caused by backhoes improperly loaded on flatbed trucks, dump trucks with their dump bodies raised, and garbage trucks with the forks up.

A vast majority of collision-induced damage where due to road vehicles (car or truck) while a minority were caused by marine vehicles. Equally a lateral force at mid-span may overcome the anchoring effects of the bearings and push the span off the piers. It should be noted that high waves in the event of a storm such as a hurricane could also impose a lateral force exceeding a critical magnitude and thus causing the superstructure to separate from the supports.

Another main cause of collapse is an overload such as a truck carrying an illegal non-permitted load. Overloading and reduced beam's cross section strength caused by corrosion, fatigue or repetitive loading may lead to beam failure. The high traffic loads that result in overloading refer to traffic loads exceeding the design load. Equally, over time, traffic growth could impose loads beyond the design capacity of the bridge. This overloading together with deterioration mechanisms (e.g., corrosion, excess heat, fatigue, etc.) can cause an imbalance in load-capacity of the bridge, thereby leading to a failure.

Deterioration of bridge components is also a leading cause of several failure cases. The main forms of deterioration threatening the integrity and safety of bridges are corrosion, fatigue, and extremely excessive heat. Corrosion is a deterioration mechanism caused by environmental factors such as exposure to water and salts, reducing the strength of the girder or pre-stressed/post-tensioned tendons and thus making them vulnerable to local buckling and normal and shear stresses.

Fatigue is a deterioration mechanism typically associated with steel members due to the cyclic loading of traffic, which results in sudden brittle failure. The strength of the beam can be reduced by temperatures greater than the maximum design temperature of 900℃. Construction errors and material deficiency also account for bridge failures.

### 2.2.2. Retaining wall failure

The failure modes considered in this section are both structural and functional. Structural failures occur in the wall segment (failure in reinforced concrete or steel sheet pile walls) while the functional failures occur due to interaction with the soil (within the retained portion of the ground).

**Figure 4. Retaining wall failure.**

Retaining wall failure modes can be categorized by:

(i) the failure of the stem due to deterioration of the concrete or reinforcement. The concrete can deteriorate by weathering, caused by e.g. freeze-thaw effects, and corrosion of reinforcement which can cause cracking and spalling of the concrete. The corrosion of reinforcement can be induced by carbonation, chloride penetration and/or combinations of both. The same failure mode can take place within a pile wall by the foregoing deterioration mechanism for reinforced concrete or by simple corrosion in the case of sheet pile walls.

(ii) the failure of the footing, which is related to the bearing capacity of the soil. The bearing capacity of the soil determines the design of the foundation footing. The greatest pressures reached are just beneath the footing; therefore, the foundation soil needs to be well compacted and soft soils (cohesive) replaced with gravel or crushed rock (granular materials).

(iii) the failure of the drainage which mostly occurs as a result of poor design of the drainage system or poor construction.

(iv) the failure of the surrounding soil which occurs due to the changes in the soil feature, leading to the failure of the whole retaining wall system. The specific failure modes are overturning, sliding, uplift by water pressure, overall instability, hydraulic heave/erosion, and the loss of bearing capacity.
For more detailed information see the Deliverable No. 2.2.

## 2.3. Risk acceptability

Infrastructure safety is often measured in terms of a structural reliability index, β, which is defined as (Davis-McDaniel et al., 2013):

$$\beta \approx \phi^{-1}(1 - P_f) \qquad (1)$$

Where $\phi^{-1}()$ is the inverse of the standard normal cumulative density function (CDF); and $P_f$ is the probability of failure. A beta value of 3.1 has been accepted by the International Organization for Standardization (ISO) as the target reliability index under ISO 2394 (ISO 1998) where the bridge failure merely led to structural damage with no casualties, corresponding to a failure probability of 1.00 × 10–3.

However, depending on the severity of the consequences following a bridge failure, also known as Consequence Classes (CC), and the resulting risk, more conservative values of beta should be employed. Table 1 shows some consequence classes and their descriptions (Lilja and Tolla, 2014).

**Table 1. Definition of consequence classes (Lilja and Tolla, 2014).**

| Consequence Class | Description |
|---|---|
| CC1 | Low consequences for loss of human life and economic; small or negligible social or environmental consequences |
| CC2 | Medium consequences for loss of human life; economic, social or environmental consequences are considerable. |
| CC3 | High consequences for loss of human life or economic; very great social or environmental consequences |

## 2.4. Failure analysis

### 2.4.1. Bridge failure analysis

Based on the causes of failure noted in Section 2.2.1, a FT has been developed in Figure 5 As previously mentioned, in order to better investigate the modes of failure the structural failure is further categorised into superstructure and substructure failures.

While the superstructure addresses the deck failure (transfer gate A), the sub-structural failures include failures of abutments /piers (transfer gate B) and foundation (transfer gate C). The afore-mentioned intermediate events (transfer gates) can further be broken down into contributing primary events, which are root causes of bridge failure. Figures 6-8 illustrate these intermediate events in more detail.

It is however worth noting that due to the presence of common PEs contributing to the failure of more than one intermediate event (e.g., the PE “Earthquake” in both deck failure (Figure 6) and foundation failure (Figure 8)), Bayesian networks (BN) can also be considered as a robust alternative to the FT approach (Khakzad et al., 2011).

Furthermore the application of BN not only facilitates the modelling of common cause failures and conditional dependencies but also makes it possible to update the risk profile as more information in form of degradation or near accidents becomes available through inspection or monitoring.

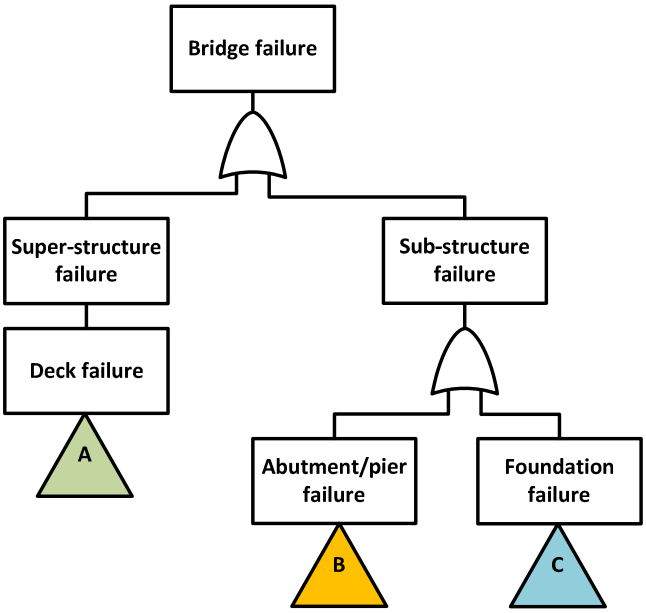


**Figure 5. Bridge failure fault tree.**

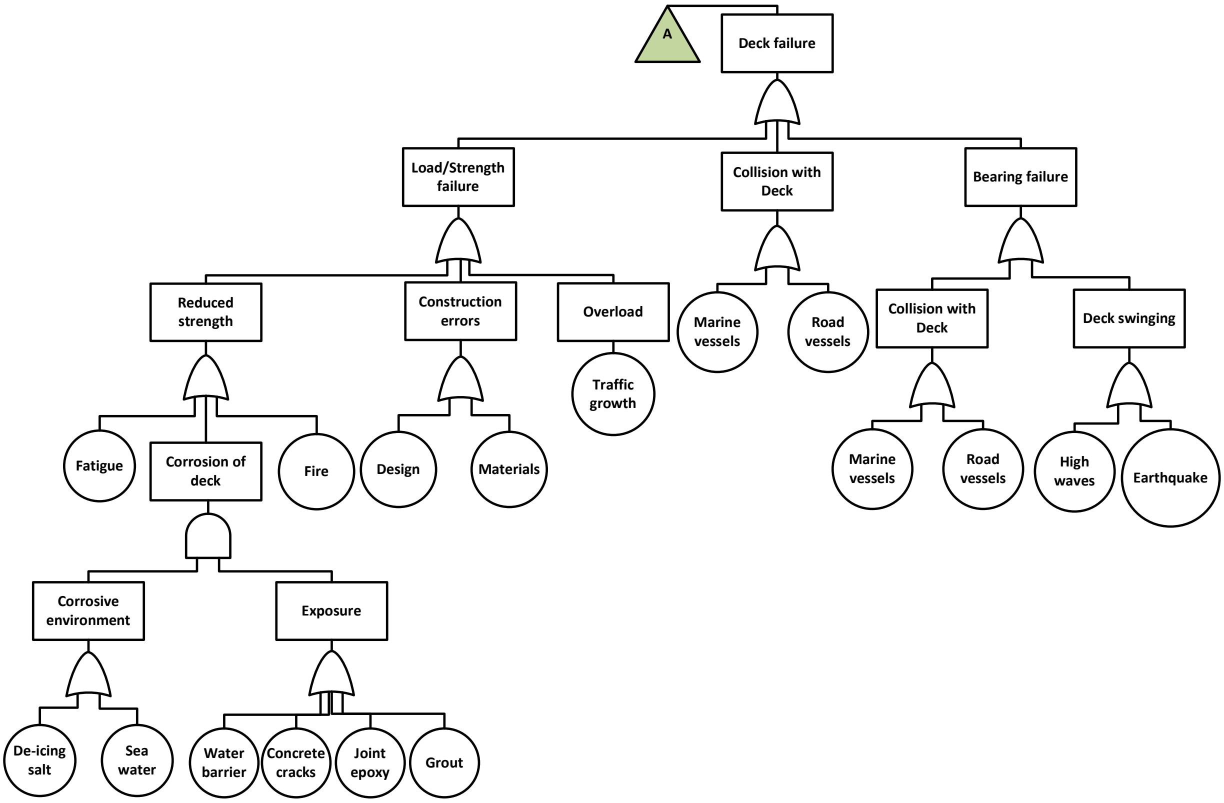


**Figure 6. Fault tree analysis of deck failure.**

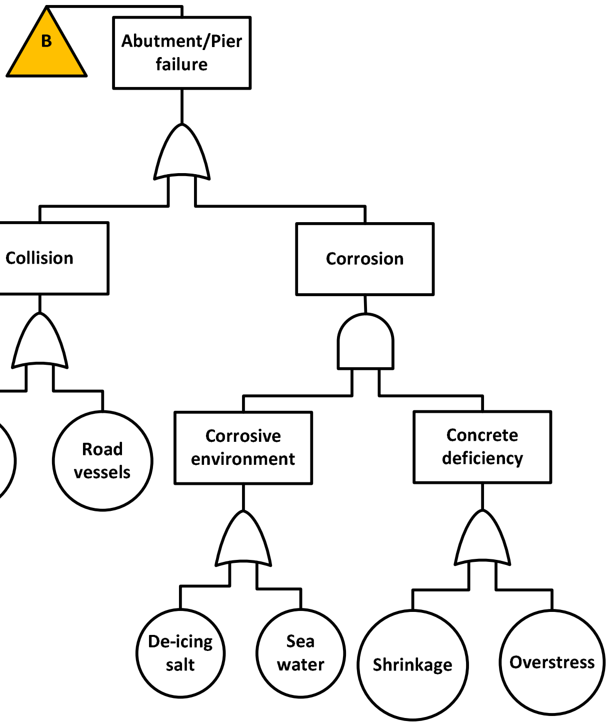


**Figure 7. Fault tree analysis of abutments/piers.**



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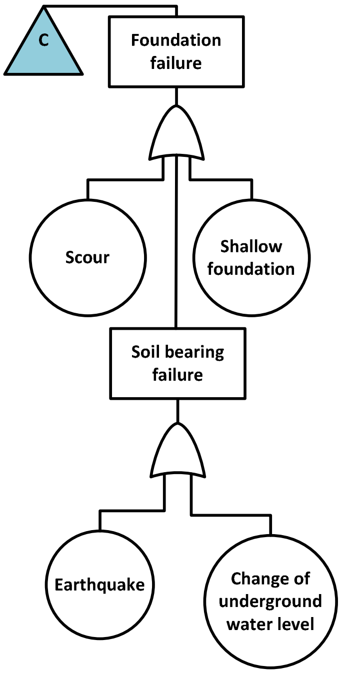


**Figure 8. Fault tree analysis of foundation.**

### *2.4.2. Retaining wall failure analysis*

Based on the failure modes described in Section 2.2.2, a FT has been developed in Figure 9.

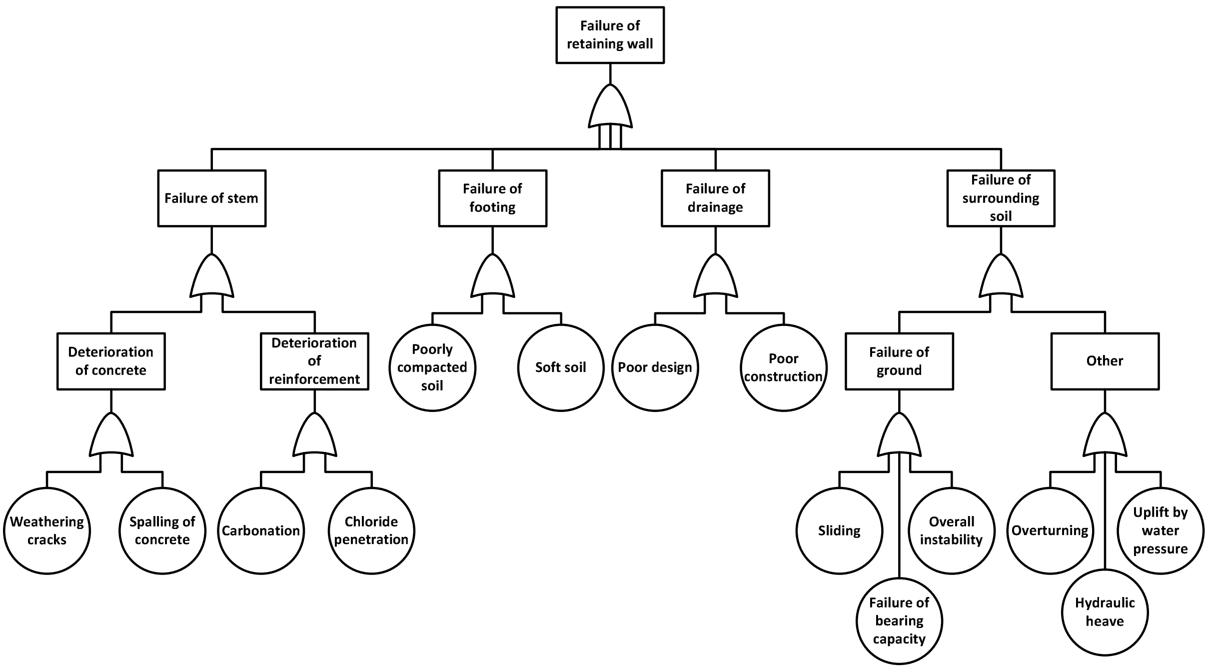


**Figure 9. Fault tree analysis of retaining wall.**

### 2.4.3. Slope failure analysis

Based on the work of Van Zyl and Robertson (1987), a FT is presented in Figure 10.

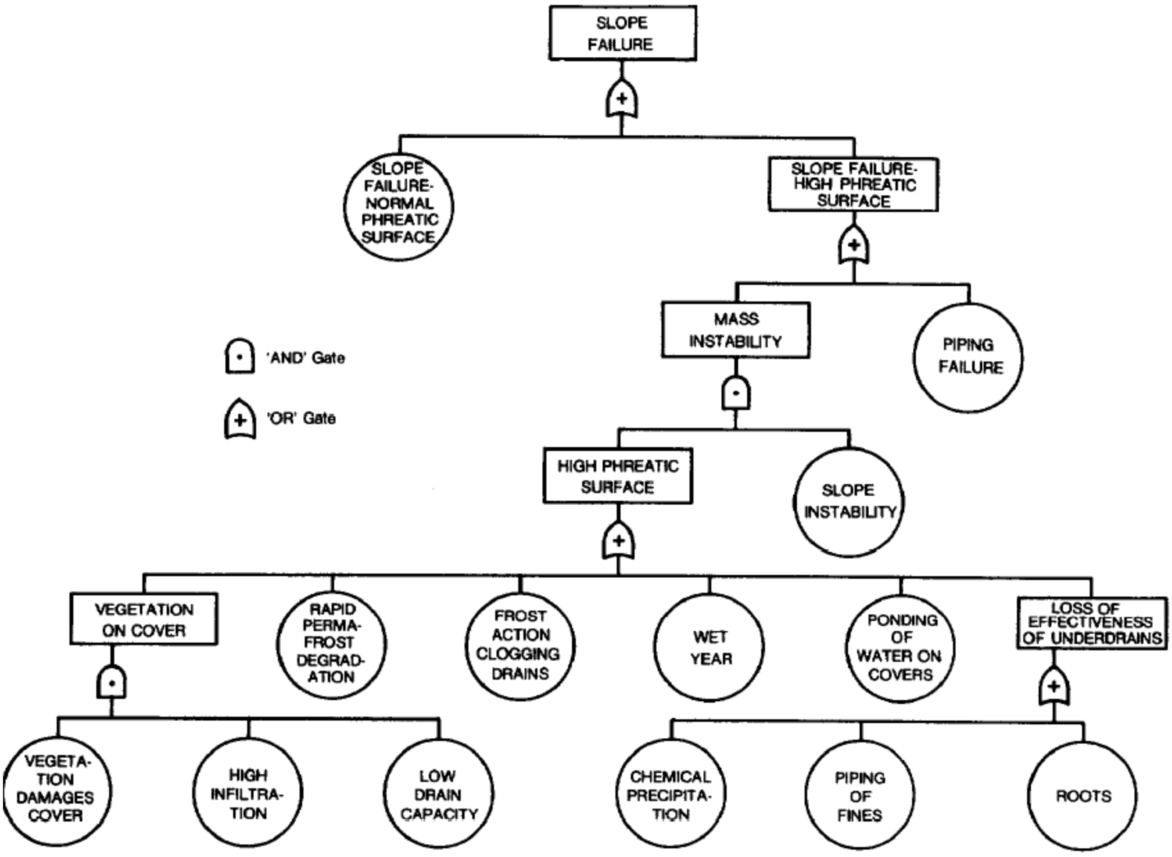


**Figure 10. Fault tree analysis for slope failure (Van Zyl and Robertson, 1987).**

## 2.5. Root cause probability estimation

As can be noted, the fault trees developed in the previous section comprises a number of primary events such as natural events (e.g., flood), man-made incidents (e.g., collision), and component failure (e.g., corrosion). These primary events can be categorized as time invariant and time dependent events.

The occurrence/failure probability of time-invariant primary events can be derived using historical data (if available) or relevant literature via the use of a meta-analysis technique (Travis et al., 2011). As an alternative, a logistic regression-analysis based on empirical data with covariates (such as age of the road section, maintenance level, traffic intensity, climate stress level, etc.) can be used as shown in Equation 2:

$$\log\frac{P_i}{1-P_i} = b_0 + b_1x_1 + \cdots + b_nx_n \qquad (2)$$

Where $P_i$ is the failure probability of the i[th] component; $x_1, \ldots, x_n$ are covariates; $b_0, b_1, \ldots, b_n$ are the regression coefficients (constants). In this model, the regression coefficients can be estimated from the literature or using actual data from road authorities (Dong et al., 2011).

The probability of time-dependent events however can be estimated using several methods:

(i) the primary events which are influenced by climate change stresses, such as floods, high winds (storms), and high waves (surges), as addressed in Work Package 2 of the Re-Gen project;

(ii) the primary events which are affected by traffic growth, such as overloading and fatigue (due to cyclic loading) can be addressed in Work Package 3 of the Re-Gen project;

(iii) for other time-dependent failures which mainly occur via deterioration (degradation) mechanisms such as corrosion can be estimated using the scoring system and Markov analysis (Orcesi et al., 2016).

For the sake of exemplification, a number of primary events, whether time invariant or time dependent, along with the methods to estimate their probabilities have been listed in Table 2.

**Table 2. Primary events contributing to fault trees of Figures 5-8.**

| Primary event | Annual probability | |
|---|---|---|
| | Time invariant | Time dependent |
| Construction errors | ✓ | |
| Flood | | Climate change (WP2) |
| Marine collision with abutment/pier | ✓ | |
| Marine collision with deck | ✓ | |
| Road collision with abutment/pier | ✓ | |
| Road collision with deck | ✓ | |
| Overload | | Traffic growth (WP3) |
| High winds | | Climate change (WP2) |
| High waves | | Climate change (WP2) |
| Fatigue | | Traffic growth (WP3) |
| Earthquake | ✓ | |
| Fire | ✓ | |
| Water barrier | | Markov analysis (Orcesi et al., 2016) |
| Joint epoxy | | Markov analysis (Orcesi et al., 2016) |
| Scour | ✓ | Climate change (WP2) |
| Tendon corrosion | | Markov analysis (Orcesi et al., 2016) |
| Reinforcement corrosion | | Markov analysis (Orcesi et al., 2016) |

# 3. Consequence analysis

In this section, economic models are investigated and derived to model the consequences of a bridge failure. These consequences can be divided into either direct costs including the reconstruction of the infrastructure or loss of human lives or indirect costs such as users' cost of travelling. Similar to failure probability estimation, the consequence analysis can also be carried out qualitatively or quantitatively.

## 3.1. Qualitative approach

The qualitative analysis of consequences resulted from a bridge failure can be performed by considering the infrastructure's safety and serviceability along with factors such as (i) the importance of the road, (ii) the traffic volume, (iii) the economic value of the bridge, and (iv) the potential consequences of a service restriction/disruption.

The importance of a road is its strategic value based on a prioritization of the national road network as very strategic roads, strategic roads, and other roads. The strategic value of the road is determined by considering the motorways or urban issues, the roads serving a strategic site (e.g. power plant, hospital etc.). The strategic value of the road can be scored and be increased by one increment to reflect the local environment, such as, a bridge crossing a high speed track).

Traffic volume measures the amount of traffic (Average Daily Traffic –ADT) over the bridge. The economic value of the bridge represents the costs of the reconstruction or repair of the bridge. The last factor characterizes the potential impact on the level of service during repairs or maintenance. Likewise, the traffic volume and the economic value of the bridge can also be scored and ranked. In the present study, however, a quantitative consequence analysis is proposed as described in Section 3.2.

## 3.2. Quantitative approach

The quantitative (monetary) consequence of road infrastructure failures includes (i) direct costs of structural damage such as reconstruction, repair, and maintenance, or damage to the life and properties of road users and (ii) indirect costs arising from the users' costs of traveling such as vehicle operating costs, travel time costs, and accident costs (Adey et al., 2003).

### 3.2.1. Direct costs

The present value cost of rebuilding the bridge structure can be calculated as the cost of the bridge per square meter of the deck surface thereof (Decò and Frangopol, 2011):

$$\mathrm{C_{Reb}(t) = C_{Reb} * W * L * (1 + r)^t} \qquad (3)$$

Where $\mathrm{C_{Reb}(.)}$ is the rebuilding cost per square meter (euro/m2); W (m) is the bridge width; L (m) is the bridge length; r is the annual discount rate of money, and t (yr) is the reconstruction time period.

## 3.2.2. Indirect costs

Several aspects of indirect costs are considered in this section such as vehicle operating costs, travel time costs, and accident costs.

### 3.2.2.1. Vehicle operating costs

Vehicle operating costs can be approximated by calculating the detour that users are forced to follow when the bridge is closed/partially closed. This is based on the duration of the detour (days or months) and the length of the detour to travel. This indirect cost can generally be described as (Deco and Frangopol, 2011):

$$\mathrm{C_{Run}(t) = C_{Run} * D_l * A(t) * d * (1 + r)^t} \qquad (4)$$

Where $\mathrm{C_{Run}}$ is the average running cost per kilometre (euro/km); $\mathrm{D_l}$ is the detour length (km); $\mathrm{A(t)}$ is the average daily traffic on year t; d is the duration of the detour (day); r is the annual discount rate of money. The time needed to restore the bridge functionality suggested in Deco and Frangopol (2011) can be established as follow:

36 months for ADT < 100;
24 months for 100 < ADT < 500;
18 months for 500 < ADT < 1000;
12 months for 1000 < ADT < 5000;
6 months for ADT > 5000.

Given that the ADT is increasing over time, costs are expected to grow over time.

### 3.2.2.2. Travel time costs

Travelling time cost, $\mathrm{C_{travel\ cost}}$, is calculated as cost for users and goods travelling through the detour (Deco and Frangopol, 2011).

$$\mathrm{C_{travel\ cost}(t) = [C_{AW} * O_{Car} * \left(1 - \frac{t}{100}\right) + (C_{ATC} * O_{truck} + c_{goods}) * \left(1 - \frac{t}{100}\right)] * [D_l * A * d]/S * (1 + r)^t} \qquad (5)$$

Where $\mathrm{C_{AW}}$ is the average wage per hour (euro/hr); $\mathrm{C_{ATC}}$ is the average total compensation per hour (euro/hr); $\mathrm{c_{goods}}$ is the time value of the goods transported in a cargo (euro/hr); $\mathrm{O_{Car}}$ and $\mathrm{O_{truck}}$ are the average vehicle occupancies for cars and trucks, respectively; S is the average detour speed (km/hr).

### 3.2.2.3. Accident costs

Consequences evaluated from the items described in 3.2.2.1 and 3.2.2.2 are only related to commercial losses. However, infrastructure failures due to the extreme weather events and traffic loads may also cause damage to human life. To assess accident costs, the methodologies developed by Zhu and Frangopol (2013) and Orcesi and Cremona (2013) can be adopted.

The French technical guidelines give the value of human life as 1 million euros (Orcesi and Cremona (2013 ). Serious and slight injuries are expressed as a percentage of the human life cost, that is 15% and 2%, respectively (150000 euros and 22000 euros).

Orcesi and Cremona (2013) indicate that the accident costs can be found for each route by applying the related accident rate. In Zhu and Frangopol (2013), however, the safety losses can be estimated using the number of casualties in a bridge-failure-induced accident and the Implied Cost of Averting a Fatality for Bridge (ICAFB). The value of the ICAFB has been suggested as 2.6 million euros.

The values of the life can be very different in different countries. There is much work devoted to the value of life (e.g., see Viscusi, 2005, and the references cited therein), which should not be interpreted as the value of any one particular life, but instead, of society's value of saving a "statistical" life. The Value of a Statistical Life (VSL) is the amount of money a person or society is willing to spend to save a life.

# 4. Risk Assessment

The risk of a bridge failure is calculated as the product of the probability of the failure and the total value of consequence of each specified failure.

$$\text{Risk} = P_f \times (C_{Reb} + C_{Run} + C_{travel\ cost} + C_{accident\ cost}) \qquad (6)$$

Where $P_f$ is the probability of failure calculated from the fault tree analysis in Section 2.
Probability of failures and consequences can be scored qualitatively or quantitatively. Accordingly, the value of the respective risk would be estimated in the same manner. Figure 8 illustrates a qualitative risk analysis.

In Figure 10, the risk has been evaluated qualitatively (or semi-qualitatively). The total consequence of a bridge failure (extent of the damage) can be ranked on an ordinal scale ranging from negligible (represented by 1) to very serious (represented by of 4) consequences.

Likewise, the probability of failure can be scored from very small (represented by 1) to very large (represented by of 4) as shown on the vertical axis in Figure 10. Based on the severity of the consequences and their corresponding probability, the value of the risk can be categorized as small (green blocks in Figure 10), medium (yellow blocks in Figure 10), or large (red blocks in Figure 10).

| | | **Consequence Severity** | | | |
|---|---|---|---|---|---|
| | | 4 | 3 | 2 | 1 |
| **Likelihood** | 4 | High | High | High | Medium |
| | 3 | High | High | Medium | Medium |
| | 2 | High | Medium | Medium | Low |
| | 1 | Medium | Medium | Low | Low |

**Figure 10. A risk matrix for qualitative risk analysis**

# 5. Risk optimization

Using a risk-based approach, it would be possible to adopt optimal maintenance/intervention strategies in order to collectively minimize the risk and allocate resources efficiently. A risk optimization methodology should be able to assist bridge owners and managers decide which bridge (or bridge components) to maintain, when to maintain them, and which maintenance strategy to take (Lounis, 2006).

Given the importance and high consequences of failure of bridges, a risk-based maintenance strategy should be adopted to optimize the risk while taking into account other objectives such as minimizing maintenance costs and user costs.

Figure 11 displays the relationship between the cost of allocated resources (horizontal axis) and the benefit (in the form of residual risk) gained from the allocation of such resources (vertical axis) in the form of a cost-benefit analysis. As can be seen from Figure 11, the optimal point at which the cost of the allocated resources (e.g., predictive maintenance strategies, retrofitting, etc.) and the gained benefit (e.g., the reduction in the amount of risk) would be equal is the interception point of the marginal cost (MC) curve and the marginal benefit (MB) curve.

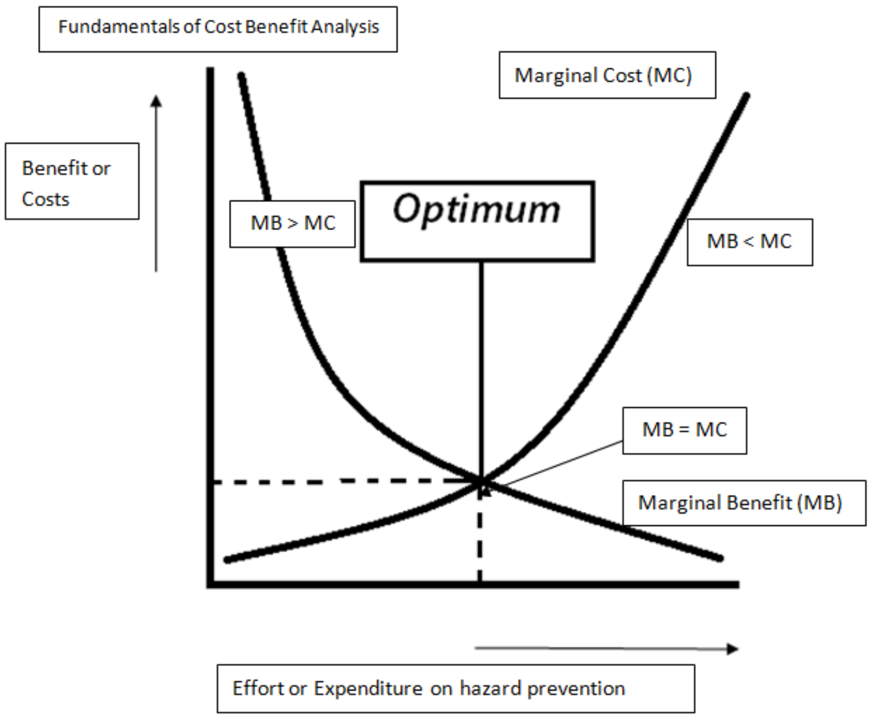


**Figure 11. Schematic of a cost-benefit analysis.**

## 5.1. Multi-attribute optimization

For the sake of clarity, consider a case where the fault tree analysis of a bridge has resulted in two critical MCSs (i.e., MCSs with the highest probabilities), that is, the failure of the deck due to the traffic growth (see FT in Figure 6) and the failure of the foundation due to scour (see FT in Figure 8), both with potentially severe consequences in terms of direct and indirect cost. It is worth noting that if a BN was used instead of FT for failure probability estimation, the concept of “most probable configuration” of events could be used to identify the most critical failure mode and thus the most critical combination of root causes (Khakzad et al., 2011).

Having the probabilities of the above-mentioned failure modes calculated from the FT analysis, the consequences thereof could be determined using Equations (3)-(5). As a result, the risk attributed to each failure mode can be estimated using Equation (6) in a monetary value. The total risk of the failure of the bridge due to these two failure modes thus can be calculated as:

$$\mathrm{R} = \mathrm{R_T} + \mathrm{R_S} \qquad (7)$$

Where $\mathrm{R_T}$ is the risk of failure due to traffic growth; $\mathrm{R_S}$ is the risk of failure due to scour; $\mathrm{R}$ is the total risk before any intervention (e.g., maintenance, implementation of safety measure, etc.).

According to the foregoing failure modes, a number of interventions can be considered, e.g., (i) strengthening of the concrete beam against bending moment generated by the traffic growth via extra reinforcement, (ii) shortening of the bridge span by adding piers, and (iii) deepening of the foundation to resist the scour. It is assumed that after the implementation of the ith intervention the value of the residual risk would be $\mathrm{R_i}$. As such, a Risk Reduction Index (RRI) can be defined for each intervention as (Yuan et al., 2015):

$$\mathrm{RRI_i} = \frac{\mathrm{R} - \mathrm{R_i}}{\mathrm{R}} \qquad (8)$$

Thus, $\mathrm{RRI_i}$ must fall between 0.0 and 1.0. The closer to 1, the more efficient the $i^{th}$ intervention is with respect to risk reduction. The RRI has the advantage of incorporating not only the direct and indirect costs of the bridge failure but also the benefit gained from the implementation of interventions.

The cost of each intervention can be estimated as $\mathrm{C_i}$, including but not limited to the cost of materials and labour. Having the total budget allocated for risk optimization B, a Cost Potential Index (CPI) can be defined for each intervention as (Yuan et al., 2015):

$$\mathrm{CPI_i} = \frac{\mathrm{C_i}}{\mathrm{B}} \qquad (9)$$

According to this definition, Ci of a suitable intervention should be between 0.0 and 1.0. If $C_i$ > 1.0; then, the cost of a given intervention is beyond the available budget, which means the intervention cannot further be considered. The closer to 0.0, the less amount of the budget the intervention costs.

Finally, Net Risk Reduction Gain (NRRG) index of the $i^{th}$ intervention is defined in this work as:

$$\mathrm{NRRG_i} = \mathrm{w_1} \times \mathrm{RRI_i} + \mathrm{w_2} \times \mathrm{CPI_i} \qquad (10)$$

where $\mathrm{w_1}$ and $\mathrm{w_2}$ are weighting factors, reflecting the preference of decision makers to whether to reduce the risk or to expend less money on interventions. It should be noted that $\mathrm{w_1} + \mathrm{w_2} = 1$. As the high cost of the $i^{th}$ intervention can make the decision maker hesitate

to apply the intervention, $CPI_i$ is considered negative in Equation (10), reducing the NRRG brought about by the intervention.
The optimal intervention strategy is the one which maximizes the sum of $NRRG_i$; that is, the net gain of risk reduction should be the greatest after the application of the optimal intervention strategy under the constraint of the limited available budget, which appears to be a typical knapsack problem. So the objective and constraint functions can be established as:

$$\text{Max } \sum_{i=1}^{3} NRRG_i \qquad (11)$$

$$\text{S.t.} \begin{cases} \sum_{i=1}^{3} C_i \leq B \\ C_i = 0.0 \text{ or } 1.0 \end{cases} \qquad (12)$$

Solving Equations (11) and (12) would result in an optimal combination of the invention strategies.

## *5.2. Decision Trees*

Likewise, a decision tree analysis can be conducted to determine the optimal maintenance strategies (Figure 12). Considering the afore-mentioned three maintenance strategies, a total number of eight maintenance alternatives (A1-A8) can be defined where in the first seven alternatives refer to the combinations of the individual maintenance strategies (i.e., A1: extra reinforcement, A2: extra piers, A3: foundation deepening, A4: extra reinforcement and extra pier, etc.) while the last alternative is to perform no maintenance.

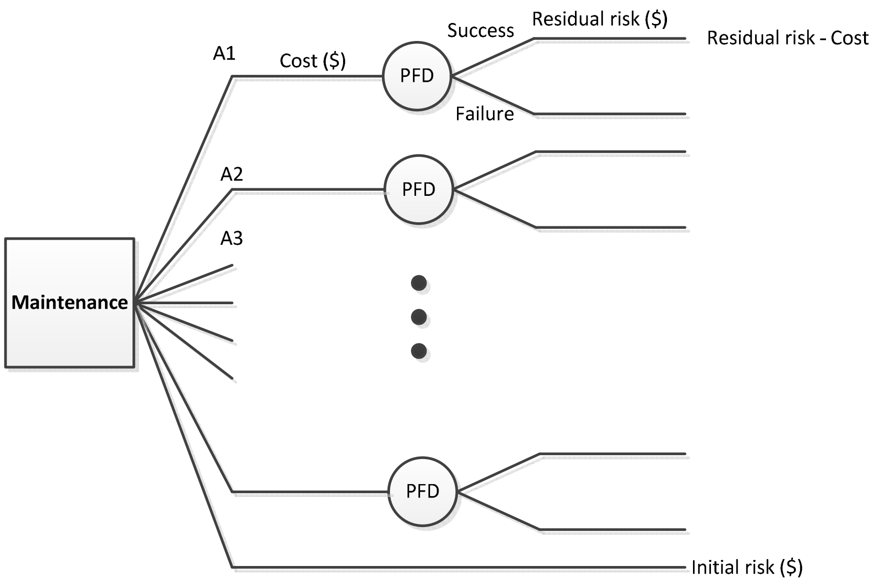


**Figure 12. A decision tree analysis for risk optimization**

The cost of each maintenance alternative (cost of materials and labour) and the respective failure probability should be determined. Assuming the success of each maintenance alternative, the value of risk can be updated by removing the corresponding failure mode from the fault trees developed in Figures 5-8.

Accordingly, the respective residual risk of each maintenance alternative would be equal to the subtraction of the initial risk and the updated risk. As such, the benefit gained from the implementation of each maintenance alternative could be calculated as the subtraction of the respective residual risk and cost (Becher, 2006).
The methodologies described in this section can be employed in Work Package 5 for risk management as well.

# 6. Discussion and concluding remarks

The objective of Re-Gen is to provide Road Owners/Managers with best practice tools and methodologies for risk assessment of road critical infrastructure elements, such as bridges, slopes, and retaining walls. As one of the goals is to prioritise critical infrastructure for maintenance and repair, the structural failure defined in this work includes structural failures (in the form of partial or total collapse) and loss of serviceability.

These failures can be caused by climate-change-induced stresses such as extreme precipitation, flood, and scour, traffic growth and infrastructure management failures (e.g. assets inspection, assets design).

In the present study, we developed a framework for risk assessment and management of road infrastructures with an emphasis on bridges, retaining walls, and slopes. We applied a fault tree analysis methodology to investigate the root causes contributing to the aforementioned failures. Developing the fault trees, we illustrated that a minimal cut sets analysis can be performed to identify the failure modes threatening the structural integrity or the serviceability of the infrastructures. In this regard, the most probable minimal cut sets can be used to short-list a set of critical failure modes.

To prevent the identified failure modes, a number of interventions (maintenance strategies or retrofitting of the structure) can be defined. Having the risk and cost indices of each intervention calculated, a risk-based cost-benefit analysis (knapsack optimization) can be performed to optimize the risk considering the cost of each intervention, the available budget, and the influence of each intervention on the value of risk (residual risk).

It, however, should be noted that owing to the presence of common root causes contributing to the failure of a bridge's foundation, piers, abutments, and deck (Figures 5-8), a fault tree methodology is likely to overestimate/underestimate the failure modes of the bridge.

To address this issue, a Bayesian network methodology (Pearl, 1988; Jensen and Nielsen, 2007) can be used (e.g. Figure 13), not only to capture the failure dependencies arising from common root causes but also to facilitate conducting a dynamic risk analysis via probability updating as new information becomes available through inspection and condition monitoring. For more information on the mapping of fault tree into Bayesian network see (Khakzad et al., 2011). It, however, should be noted that in case of a qualitative risk assessment, a FT methodology would provide more information about the MCSs since the logical relationships among the primary events are readily reflected via AND/OR gates.

The developed Bayesian network can further be extended to a limited memory influence diagram (LIMID) by adding decision and utility nodes. A decision node can include a number of decision alternatives each representing an intervention strategy while a utility node comprises a number of utility values representing the preferences of a decision maker regarding each decision alternative (Gilboa, 2009).

The utility values can be determined using utility functions developed based on the available budget, the cost of each intervention, and the impact of each intervention on the value of risk (in a risk-based cost-benefit analysis). A recent application of LIMID to risk optimization can be found in (Khakzad and Reniers, 2015).

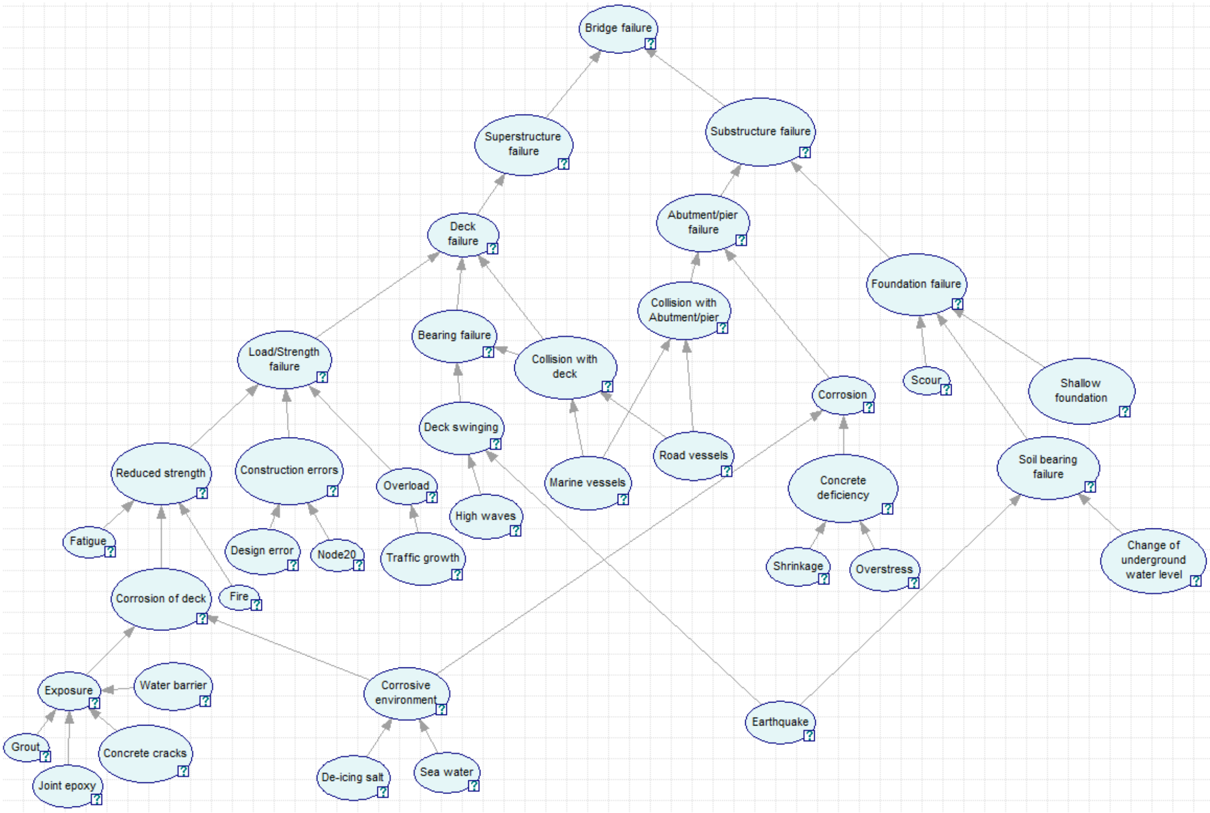


Figure 13. Failure assessment of bridges using Bayesian network

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