EVALUATION OF THE RESIDUAL LOAD-BEARING CAPACITY OF CIVIL STRUCTURES USING FUZZY-LOGIC & DECISION ANALYSIS

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A	Frontal Area, Cross-Section Area
$A(t)$	Cross-Section Area Remaining after Time t
B	Building Breadth (Ductility and Dissipation Capacity)
C	Costs (Cost Minimization), Confirmation Degree
$C_{repair,i}$	Repair Costs
C_{Vi}	Confirmation Degree for Cause no. i
$C(\beta)$	Increasing Costs dependent on Safety Factor
C_{total}	Total Costs
a_i, b_i, c_i	Unknown Coefficients of the First Order and Second Order Polynomial Equation (Limit State Function for Risk Analyses)
c	Light Velocity, Normalizing Factor for Baeysian Function
D	Damage Index
D_w	Weighted Damage Index
D_i	i^{th} Damage Index contributing to D
D_{max}	Maximum Damage Indicator
D_{fire}	Index for Fire Damage
$D_{stephen}$	Stephens‘ Extended Index (Damage Index)
Δd_p	Positive Change in Plastic Deformation (Stephens Index)
d	Diameter, Effective Depth for Cracked Sections
d_0	Initial Diameter

$d_{reflect}$	Distance of Reflecting Surface (Geometry Recording)
$d_w(V_i, V_j)$	Weighted Hamming Distance
E	Elastic Modulus, Energy, Independent Equilibrium Equations (Ductility and Dissipation Capacity)
E^*	Error Discrepancy (Neuronal Networks)
E_0	Initial Elastic Modulus
E_d	Secant Elastic Modulus
$EI(t)$	Bending Stiffness of Damaged Beam (Damage Index)
EI_0	Bending Stiffness of Undamaged Beam
E_i	Dissipated Energy at Spring i
$E(B)$	Expected Benefit
e^{α}	Pressure Rate Decay
F	Impulse Integral, Force
F^*	Limit Enhancement Parameter
f_{st}	Ultimate Stress
f_y	Yield Stress
f_T	Limit State Function dependent on Elapsed Time T and on $g(x)$
G_f	Fracture Energy
$g(x)$	Limit State Function (Risk Analysis)
g_R	Function of Empirical Limit Data
g_S	Function for the Actual Force Distribution
g_m	Mean Value of Limit State Function
H	Building Height, Importance Matrix
h	Total Concrete Depth for Uncracked Section

h_{ij}	Singletons for Relative Importance of Damage Effects
I	Moment of Inertia, Indeterminacy
I_{uncr}	Moment of Inertia of Uncracked Section
k_c	Concrete Corrosion Rate
k_s	Steal Corrosion Rate
k_0	Initial Tangent Stiffness (Damage Index)
k_f	Secant Stiffness
k_{max}	Secant Stiffness at Maximum Deformation due Applied Loads
$L_{Mitigation}$	Loss Reduction from Mitigation Measures
$L_{Building}$	Building Length
L_{dam}	Ultimate Strength of Damaged System (Dissipation Capacity)
L_{int}	Ultimate Strength of Intact System
L_i	Prognostic Factor
$L(\Theta)$	Likelihood Function based on Analytical and Experimental data (Objective Information in Baeysian Approach)
l	Length of Structural Component, Story Height
l_{cr}	Critical Length of a Crack
l_p	Length of Structural Component over which Steel Stress exceeds Yield
M	Mass, Moment
M_{cr}	Cracking Moment
$m(x)$	Mean Value
N	Load, Number of Rebars / Polynomial Coefficients / Samples
N_{yield}	Load at Yielding

N_{cr}	Global Critical Load
N_{polyn}	Number of Polynomial Coefficients (Risk Minimization)
n	Number of Random Variables, Number of Building Stories
O_i	Actual / Calculated Output (Error Minimization)
O_{neuron}	Actual / Calculated Output (Neuronal Networks)
P	Pressure, Probability
P_r	Reflected Pressure
P_{so}	Side-On Overpressure
$P(t)$	Forcing Function of Dynamic Blast Load
$P_{costs,i}$	Probability for Repair Costs
P_{det}	Detection Probability of a Fault
P_f	Risk for Whole System
K	Correction Vector reflecting Degree of Agreement
$P_{target,f}$	Target Risk
$P_{f,stat}$	Failure Probability (Calculated from Density Function)
p_i	Component Risk
k_i	Singletons for Correction Vector K
$p(\Theta)$	Prior Probability Density Function for Initial State of Expert Knowledge (Subjective Information in Bayesian Approach)
p_{old}	Annual Probability of Disaster without Mitigation Measure
p_{new}	Annual Probability of Disaster with Mitigation Measure
Q	Performance Index (Uncertainty Consideration)
q_i	Damage Effect i

R	Distance Explosion Center/Target, Distance Stiffness/Gravity Center, Redundancy, Resistance
$R(x)$	Resistance-Deformation Curve
$R(t)$	Resistance-Time Curve
R_e	Maximum Lateral Load Resistance
R_{deter}	Deterministic Redundancy
R_{prob}	Probabilistic Redundancy
r	Number of Realizations, Annual Discount Rate
T	Natural Period, Relevant Time Horizon
$T_{i,initial}$	Eigenperiod of Undamaged Structure
$T_{i,max}$	Eigenperiod of Damaged Structure
T_i	Desired Output (Error Minimization)
T_{neuron}	Desired Response (Neuronal Networks)
t, T	Elapsed Time
Δt	Time Interval of Reflection (Geometry Recording)
t_{crit}	Critical Time
t_d	Explosion Duration
u_{max}	Displacement Maximum of Inelastic System
u_{yield}	Displacement at Yielding
$u_{failure}$	Displacement at Failure
V_m	Strain-Rate with respect to Dynamic Strength
V_k	Loading-Velocity with respect to Static Strength
V_i	Damage Cause No. i
W	Explosive Weight

W^*	Shape Parameter, Weighting Vector (Fuzzy-Set)
w_i^*	Singletons of Weighting Vector
dw	Opening Displacement
w_i	Weighting Value (Damage Index Calculation)
w_{old}	Preliminary Connection Weight (Neuronal Networks)
w_{new}	Updated Connection Weight (Neuronal Networks)
Δw	Calculated Error added to Preliminary Connection Weight
x	Neutral Axis Depth, Damage Level
x_i	Variable in Limit State Function
x_e	Elastic Deformation
x_p	Plastic Deformation
x_{stat}	Static Deformation
$x_{dynamic-max}$	Maximum Dynamic Deformation
$x_{impulsive-max}$	Maximum Impulsive Deformation
x_r	First Deterministic Mean Approximation dependent on ξ
y_r	Distance Cross-Section Centroid / Extreme Tension Fiber
z	Scaled Range
α	Learning Rate
α_{crack}	Coefficient for Crack Length (Fracture Analysis)
$\alpha_{fatigue}$	Fatigue Exponent (Calculation of Damage Indices)
β	Strength (Fracture Analysis), Safety Index (Risk Analysis)
β_D	Compressive Strength of Concrete
β_{stat}	Static Strength

β_{dyn}	Dynamic Strength
$\beta_{m,dyn}$	Dynamic Strength at Strain-Rate V_m
$\beta_{k,stat}$	Static Strength at Strain-Rate V_k
$\beta_{syst-coll}$	Safety Index of Intact System with respect to Collapse
β_{weak}	Safety Index of Weakest Member with respect to Failure
$\beta_{stories}$	Factor dependent on Number of Stories
β_{Park}	≈ 0.25 (Park and Ang's Damage Index)
β_{rot}	Constant of Integration
$\beta_{c,axial}$	Post-Yield Stiffness Coefficient
δ_s	Shear Deformation
ε	Material Dielectricity, Strain
ε_u	Rebar Strain at the Imposed Loading
ε_y	Rebar Strain at Yielding
$\frac{\delta\varepsilon}{\delta t}$	Reference Strain-Rate
$\frac{\delta\varepsilon^*}{\delta t}$	Dimensionless Strain-Rate
ϕ	Probability Density, Distribution Function of Standard Variable
γ	Coefficient dependent on Information Type, Significance and Reliability (Weighted Damage Index Calculation)
γ	Ratio between Applied Axial Load and Global Critical Load
λ_1	Upper Limit to Excess Strength of Element 1 beyond which, for given Post-Yield Strength of 2, it can never Yield
η	Weighted Damage Extent
η_{ultim}	Damage Extent at Limit State

$\eta_{0.5}$	Minimum Damage Extent
$\eta(t)$	Damage Extent at Time t
σ	Post-Yield Stiffness Coefficient
σ_v	Axial Compressive Stress
σ_{insp}	Parameter for Inspection Technique/Scheduling/Quality
$\sigma(x)$	Standard Deviation / Coefficient of Variation
σ_m	Standard Deviation of Basic Variable x
$\sigma_{t,max}$	Maximum Tensile Strength
θ	Rotation Angle [rad]
θ_p	Plastic Rotation
$\theta_{failure}$	Hinge Rotation at Failure
θ_{max}	Hinge Rotation
μ_{rot}	Rotation Ductility
μ_c	Curvature Ductility at Plastic Hinge
μ_d	Displacement Ductility
μ_Δ	Upper Limit for System Ductility
$\mu_{\Delta 2\,max}$	Max Ductility Demand expected to develop in Element 2
$\mu(DE_i)$	Trapezoidal Membership Function
μ_{ij}	Singletons defining Relation Space
Θ	Unknown Parameters (Baeysian Approach)
ψ	Geometric System Parameter (Unrestrained Torsion)
ω	Eigenfrequency
ξ	Probabilistic/Random Uncertainty, Variable (Sensitivity)

1 Introduction

There has always been a need in the evaluation of civil structures after extreme events. In the past natural catastrophes such as earthquakes, flood or fire frightened the population. Today mankind additionally is confronted with terrorist and military attacks. Therefore engineers do not only have to design new buildings, but also evaluate existing civil structures to help stakeholders in deciding if these have to be destroyed or if these can serve for partial use. They are confronted with extensive and wide-ranging tasks requiring a cooperative effort of several disciplines. Only an interdisciplinary effort involving the expertise of experts can guarantee a realistic evaluation. Relevant disciplines for condition assessment are illustrated in Figure 1-1:

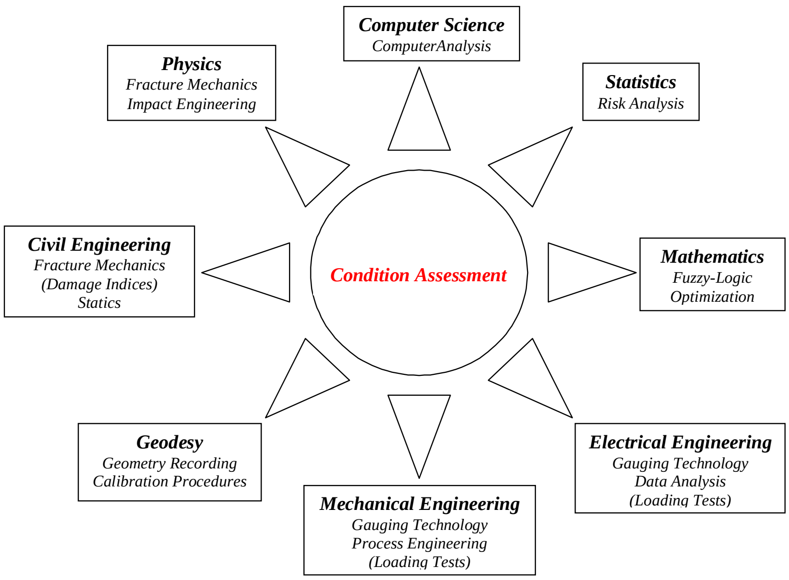

Figure 1-1: Disciplines for Condition Assessment

1.1 Actual Situation in Condition Assessment

Fracture mechanics and computational analyses methods, techniques for geometry recording and material investigation have been considerably improved in the last decades. To a lower extent this is also true for research fields such as ductility, energy dissipation and risk analysis (chapter 2). The collected knowledge has been used to develop procedures for an actual scheme of the evaluation procedure: After having defined appropriate damage indices the recorded data are prepared and transformed allowing for subsequent analyses. Modeling and model updating require numerous research activities to complement parametric techniques by nonparametric ones such as neuronal networks (chapter 3).

1.2 Question proposed for Solution – Motivation for Current Research

Since the present methods have been developed for analyses during the erection of civil structures, their application for existing ones does not always lead to acceptable results. First, the data are not poured in continuously and completely. Second, they can be misunderstood, since one cannot assume a static system to be unknown in the same way as the imposed loads. Nevertheless, experts carry out condition assessment proceeding with the same methodology that is used in draft and design. Often they base their decision on single analysis results or pictures. Although both tasks are completely different, economical and time related reasons make the engineers still apply them. However, existing structures need a different approach, capable of identifying their system dependent on the imposed loads and its response to them.

1.3 Goal of this Publication

Owners and stakeholders are not satisfied with such a subjective evaluation and expect that uncertainties are made evident and not hidden. Not only, that they fear a lack of safety, but also do they want an objective advice with respect to necessary monetary, material and personal expenses for a future decision about repair. The actual deficiencies of evaluation procedures are obvious:

- no objectivity since type and location of the failure modes are unknown
- no systematic fault analysis so, that many problems are overseen
- negligence of uncertainties in geometry, material, loads or modeling
- probabilistic analyses pretend to be reliable, although their request of sufficient data is not fulfilled.

Therefore the purpose of this paper is first to show, how random research for structural defaults can be avoided by investigating systematically possible failure modes with the help of a fault tree. In this way details are verified and no important aspects overseen. In a comprehensive methodology screening assessment, approximate evaluation and further investigations are performed in three subsequent steps. Whereas in the screening assessment a rapid classification for planning immediate precautions to save lives and high economic values is intended, in an approximate evaluation a structure is analyzed more in-depth. For the case that, due to complex failure mechanisms, a final decision about what to do is impossible, further investigations should reveal the reality of the structure with respect to its exact geometry and material properties. Exhaustive computational analyses based on fracture mechanics and dissipation capacity are performed to decide about destruction or repair. If the cause-effect relationship of damage patterns cannot be quantified due to e.g. vague multi state characteristics or uncertainties in material, geometry, loads or modeling, the dominant failure mode may be determined using calculations based on fuzzy-sets.

2 Fundamentals for Post-Incident Investigation

2.1 Introduction

The reader will be introduced in -for a structural evaluation- necessary disciplines, mainly fracture mechanics, computational analysis, geometry and material investigation, ductility and energy assorption, risk analysis, cost optimization and uncertainty consideration.

2.2 Theoretical Basics for Damage Analysis

Fracture mechanics may be defined as the part of solid mechanics and material science which deals with the behaviour of cracks. It relates global forces and deformation to local fracture behaviour with respect to the crack front in order to describe the mechanic response of damaged structural components. Fracture mechanics use micro-mechanic 3D models monitoring crack propagation with variable strains to exploit the utmost cumulative material ductility. Although there are some refined theories, generally material parameters are described on macroscopic level. Whereas the constitutive behaviour of a metal continuum may be identified in meso-scale by a micro mechanic model, for the constituents of concrete and masonry this is possible only for their initial elastic behaviour. Then fracture induces local inelastic deformation at micro-level (micro-cracks initiate at about 60% of the maximum tensile strength) together with continuous deformation and diffuse distress at macro-level, macro-cracking and debonding being a small volume compared to any deformed body (Hofstetter and Mang; 1995, Amara, K. B.; 1996 and Sha, S. P. et McGarry, F.; 1971).

Here, inelastic deformation do not only result from plastic slip, but also from continued micro-cracking at higher stress levels leading to a decrease of the moment of inertia and to some degree of the elastic modulus. The relevant strength during failure is not defined by the peak point, but by the ultimate point on the complete relationship, where inelastic strains and yield surface distortion change the texture of material. Strain softening with negative stiffness refers to the descending part of a stress strain curve. Generally, it is appropriate to consider load-induced damage due to elasto-plastic yielding, low-cycle fatigue or rupture of reinforcement steel, crushing after compression, spalling after tension and, time-induced damage due to material degradation due to hysteresis or material damping, creep, shrinkage, wear and tear, chemical and mechanical attack) at the most severely stressed locations (Chen, Z.; 1996, Krätzig, W.B.; 1999 and Krätzig, W.B.; et altri; 2000).

Stress distribution in concrete depends on the aggregates' stiffness and on the matrix porosity. Reinforced concrete, however, is governed by the interface behaviour between concrete and rebars. This depends on bond modeled by a tension stiffening[1] concept and, on dowel action in models generally neglected. Bond refers to the longitudinal force transmission from concrete to steel in case of adequate anchorage. Insufficient bond leads to a de-coupling of concrete or de-bonding, where the compatibility of deformation between steel and concrete can no more be maintained (see Figure 2-1). Here, a new crack tip is no longer orientated normal to the applied tensile stress.

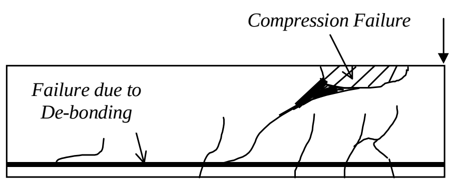

Figure 2-1: Failure due to De-bonding and Compression

Dowel action refers to an increased shear force transmission across cracks intersected by rebars (see Figure 2-2). If these are pulled, additionally to friction circumferential forces in the surrounding concrete produce large compression at moving steel ribs being comparable with hydrostatic pressure acting on a thick-walled concrete ring. Here, aggregate interlock from rough crack surfaces causes tensile rebars to act as dowels.

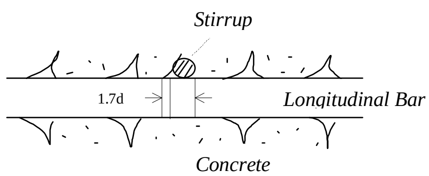

Figure 2-2: Dowel-Action

[1] interaction between the stirrups and concrete

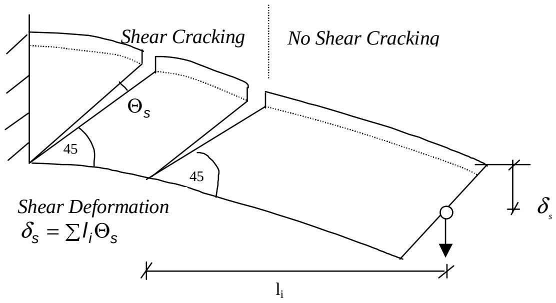

Figure 2-3: Shearing Failure of a Cantilever Beam

Micro-cracking in the softening part of the stress-strain curve, crack-coalescence and macro-crack formation prevent that tensile stress is transferred. Dependent on fracture toughness (resistance to crack extension) a ductile failure mode can change into a brittle one such as shearing (see Figure 2-3). Beyond the elastic limit, a bond crack propagates in a stable manner up to discontinuity stress with stress not immediately falling to zero, but decreasing with increasing crack width. Large irreversible strains at relatively low stresses contribute in forming a damage zone as long, as there is a continuous energy supply. The propagation stops at bond cracks, unbroken ligaments, between macro-crack segments, micro-voids, air holes, or at lightweight aggregates. Joining with bond cracks micro-cracks are forming continuous cracks parallel to the loading path (Hofstetter and Mang; 1995). Unloading only leads to stabilization if crack widths are below 0.05 mm and, if real crack lengths are shorter than critical length[2]

$$l_{cr} = 600\alpha_{crack}\left(\beta_D\right)^{-0.3} \qquad (1)$$

Brittleness of concrete structural response is intimately connected with size. An increase in the aggregate size from 8 mm to 16 mm or 32 mm changes the value for α from 4 to 6 or ten, respectively. Another measure for brittleness is the fracture energy,

$$G_f = \alpha_{crack}\left(\beta_D\right)^{0.7} \qquad (2).$$

Ductility in the same way as brittleness depends on inertia forces, damping, mean compressive strength β_D (also being related to aggregate size) and flaw density (Karihaloo, B.; 1996 and Könke, C.; 1999c). The higher the aggregate proportion, the higher fracture toughness is and, the later cracks initiate at material discontinuities (see Figure 2-4 and 2-5). Although crack density depends on cement type, coarse aggregate texture and water cement ratio, in an

[2] size of the crack when failure is expected to occur

engineering approach concrete composition can be neglected considering only concrete and reinforcement grade.

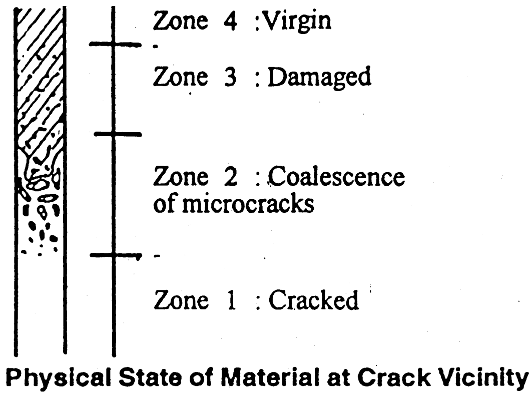

Figure 2-4: Crack Propagation (Amara, K. B.; 1996)

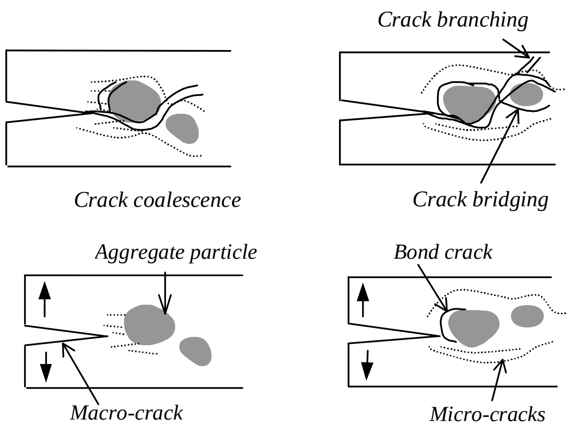

Figure 2-5: Crack Type and –Development (Karihaloo, B.L.; 1996)

Compared to non-model-based methods assessing damage straight-forward by comparing data from structures before and after a damaging event, signal or model-based methods locate and quantify damage by correlating analytical models with test data of the damaged structure. Actually several model types are in use (Seeger, T. et altri; 1999, CEB; 1988 and Sanjay, G.et altri; 1997):

- entire loading scenario under the relevant boundary conditions,
- structure considering member geometry, material (constitutive laws) and discretization dependent on workmanship or detailing
- stress concentration, energy absorption, debonding and hinge formation

- transformation analysis considering the relationship between existing and allowable stresses or deformation.

Up to now there is no generally accepted constitutive law for concrete. Most of the published models are characterized by acceptable deviations between numerical and experimental data. The problem is, that the behaviour during e.g. crack closure ranges from brittle to ductile. It therefore requires a unified treatment of compression and tension with constitutive relations for intact and damaged material. Whereas failure envelopes associated with the ultimate strength are easy to produce, those associated with the ultimate strain may not be proved by experimental results (Hofstetter & Mang; 1995).

An elastic damage model using measurements of local stress in critical areas might replace a previous application of separate models for loading and structure. Here specified stress-strain relations at least approximately describe in-plane structural members such as walls or columns being large compared to crack spacing or fracture process zone (3-5%) relative to crack length and width (Roca, P. et altri; 1997 and Bazant, Z.P.; 1997). The theory of plastiticity describes in-homogenity due to different strength of concrete components or low contact strength between cement matrix and aggregates even considering the size-effect[3] with its steep softening part, strain localization, crushing and steel hardening (Bracci, J. M. et altri; 1997). All these phenomena having been modeled on material level then are homogenized to the structural level.

Homogenous bodies with continuous failure modes do not need elements to cover each joint individually. Reinforced concrete, however, is best discretized with elements that span only over one material introducing zero-thickness interface elements along the crack path, at joints and along adjacent nodes of concrete and steel elements. The different layers' stiffness contribution can easily be integrated into the stiffness matrix of finite elements. Their variation throughout the section is no problem. Non-geometrical constitutive or kinematic models reduce the stiffness or shear modulus neglecting a possible displacement discontinuity at nodes. Micro-cracks are averaged in the neighbouring material with the result of macro-mechanic values. So, the "smeared crack approach" or "macro-element approach" (experimental) imagines a cracked solid to be a continuum with a fixed topology of the FE-mesh.

Whereas in a distributed representation interface effects are not discretized, the "discrete crack approach" or "microelement approach" (analytical) models a crack as a geometrical discontinuity. It describes a strain field from a discrete set of displacements and their relation to the respective forces across the crack. Here, a dense grid of small finite 3D elements for concrete and bar elements for

[3] induced either by forces with different physical dimensions (finiteness of specimen sizes), by an imperfection within the matrix or at the interface between matrix and aggregate (aggregate interlock), discontinuities in the flow of the stress

steel, both continuously re-meshed during crack propagation, allows modeling crack energy (Rots, J.G. et altri; 1989, CEB; 1988 and Chen-Man, C; 2000).

FE analyses applying a point-wise description of the continuum try to reproduce the experimentally observed behaviour. They are appropriate only to verify a few critical areas or, to perform a corrector analysis to consider bond and dowel action. Constraining the crack in a predefined path along element edges would not fit the nature of FD analyses primarily using an element-wise description. Differences to numerical values for the ultimate load and the corresponding deflection are caused by an underestimation of the residual stiffness of cracked concrete or of tension stiffness. The relationship between loading and structural response is illustrated in Figure 2-6.

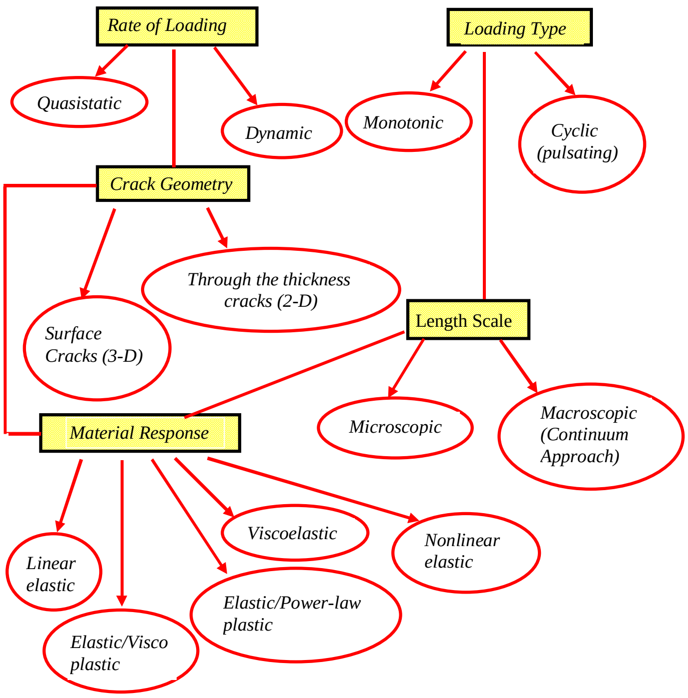

Figure 2-6: Relationship between Loading and Structural Response

The redistribution of internal forces may be determined by simultaneously (not separately!) subjecting all structural components to the following analyses (Karihaloo, B.; 1996):

- Superficial damage with small cracks only requires a general stiffness reduction (elastic).
- Moderate damage with distributed fracture allows a smeared crack approach of linear fracture mechanics reducing the stiffness of only one unit in the FE mesh (elasto-plastic)

- Extensive damage with few major cracks, where debonding and interlock are significant, requires a discrete crack approach of nonlinear fracture mechanics (plastic)

Mixed states of tension and compression stress should be investigated avoiding an undesired interaction between specimens in tests and reality due to a frictional restraint or size effect. Energy absorption is similar to the plastic flow in metals and may be described by the thermodynamics of irreversible processes with constant energy release per unit area (Hillerborg, A. et altri; 1976, Carpinteri, A. et altri; 1983 and Curbach, M.; 1987). The use of the fracture energy

$$G_f = \int \sigma_{t,max} dw \qquad (3)$$

as additional parameter guarantees mesh independence (Menetrey, Ph.; 1997). In this Griffith formula the variable $\sigma_{t,max}$ represents the maximum tensile strength dependent on crack opening displacement w_r from zero to rupture. The fracture energy or toughness of a material is equivalent to the amount of energy (proportional but not equivalent to stress!) required to break a given cross-section. This is not the same as the tensile strength. As measuring the external dissipated fracture energy (vibration, noise etc.) is subjected to numerous uncertainties, either the internally stored and, irrecoverable hysteretic part (material yielding or damping) is used to describe damage, or the total energy transmitted to the structure representing the area under the stress-strain curve multiplied by the volume of the element.

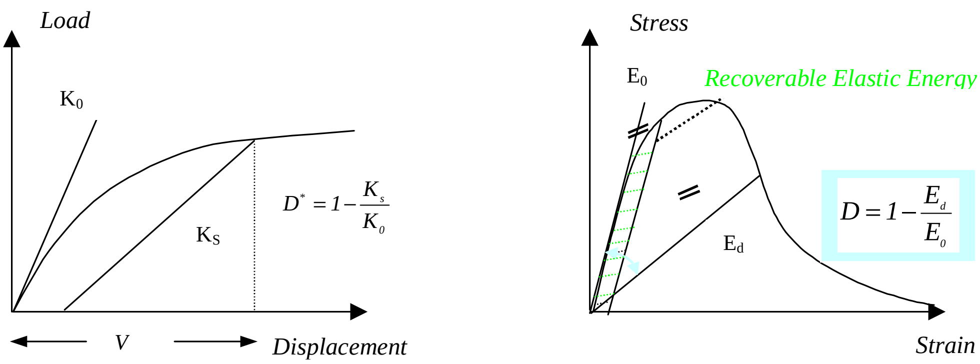

Figure 2-7: Damage Treatment on Structural and Material Level

Hardening results from the magnified failure surface due to crack opening after unloading and is defined in terms of work performed during a loading/unloading cycle before the ultimate stresses are reached. Here both inertia forces and moisture content in the pores lead to a strength and stiffness increase. Softening, however, results from negative stiffness in the end of the stress-strain curve (large deformations and damage on material and structural level). Geometric and kinematic equations relate nodal forces to element deformation considering symmetry requirements (large external forces disturb the static equilibrium and increase displacements). Supposed that the material parameters are sufficiently homogenous and experimentally measurable, integral results are interpolated converting microscopic to macroscopic stress/strain (see Figure 2-7).

Limit situations are pure tension with zero shear stresses (Mode 1 being planar symmetric and normal to the fracture surface) and shear or sliding with very high compression due to a sudden release of stored energy (Mode 2 being planar anti-symmetric and tangential to the fracture surface, but normal to the crack front). This highly brittle mode leading to crushing (more dangerous than tensile fracture) is sufficiently homogenous, where softening is improbable and so, may be uncoupled from Mode 1 (Hofstetter and Mang; 1995, and Karihaloo, B. L.; 1996). Torsion about a normal to the fracture surface lead to deformation induced tearing (Mode 3 being out-of-plane anti-symmetric, tangential to the fracture surface, but parallel to the crack front). Prevalent failure modes and the corresponding load-deformation curve are shown in Figure 2-8.

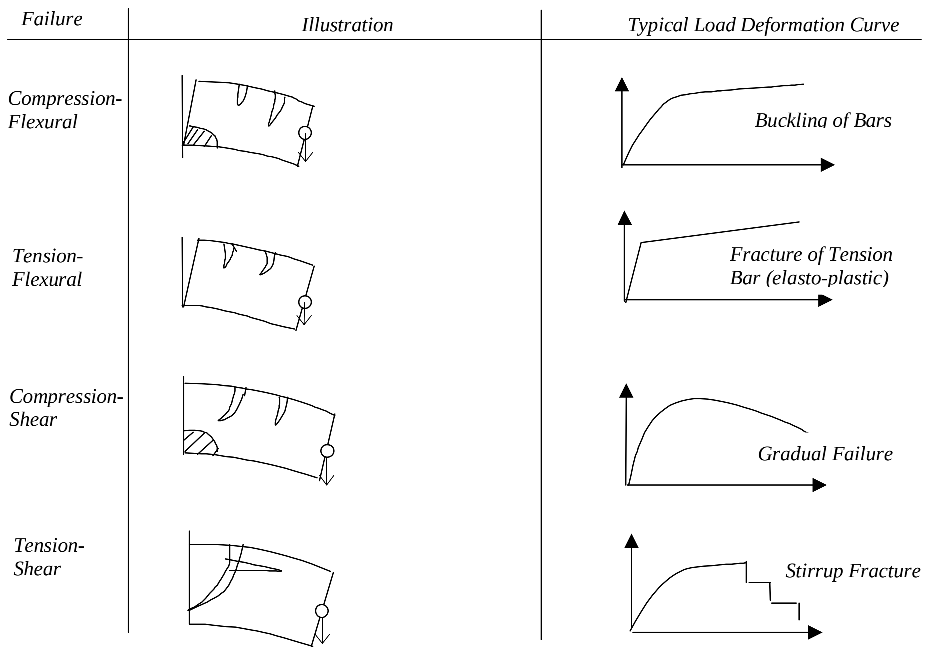

Figure 2-8: Prevalent Failure Modes, Load-Deformation Curve (Park, R; 1996)

Whereas tension stiffening may be modeled quite well, the problem of dilatancy at high-rate straining is still subject of current research (Perry, S.H.; 1999). Up to 10^3 cycles the fatigue strength is decreasing at a much higher rate than is the case above 10^4. In the low cycle region, failure primarily occurs because of matrix-cracking cycles near the discontinuity stress. For high-cycle loading, failure generally is caused by bond cracking. For strain-rates $> 10^2 \frac{1}{s}$ and pressures $> 6Pa$ lead to an inelastic response (dilatancy -volumetric expansion!)

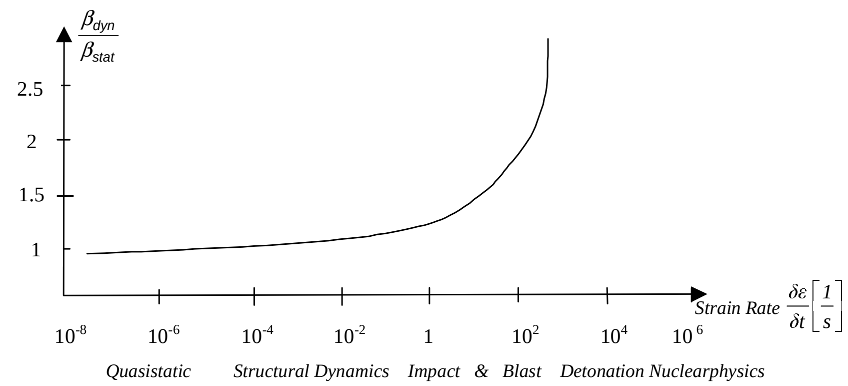

Figure 2-9: Relation between strain rate and dynamic strength (Ruppert, M. et altri; 1999)

Even trilinear enhancement relations would lead to an overestimation of strength dependent on the loading history and on the strain-velocity V_m (for strain rates $> 10^3/s$ no experimental results have been published yet, Curbach, 1987; CEB Bulletin, 1987 and Ruppert, M. et altri.; 1999). The actual dynamic strength $\beta_{m,dyn}$ may -dependent on the static strength $\beta_{k,stat}$ at a loading velocity of V_k - be formulated with (Specht et altri; 1996).

$$\beta_{m,dyn} = 0.1\beta_{k,stat}\left(log\frac{V_m}{V_k} + 10\right) \qquad (4)$$

However, here the different behaviour of brittle concrete in tension and compression is neglected. Another formula allows considering as well the reduced quality of concrete changing from solid to granular material after the compaction of voids and pores (see Figure 2-9):

$$\beta_{dyn} = \beta_{stat}\left\{0.4\left[\tanh\left(\log\frac{\delta\varepsilon^*}{\delta t}\right) - 2\right]\left[\frac{F^*}{W^*} - 1\right] + 1\right\}W^* \qquad (5)$$

Herein $\frac{\delta\varepsilon}{\delta t}^{*}$ represents the dimensionless strain-rate normalized with respect to a reference strain rate $\frac{\delta\varepsilon}{\delta t} = 1.0\frac{1}{s}$ and F^{*} the limit enhancement parameter for $\frac{\delta\varepsilon}{\delta t}^{*} \to \infty$ running from 3.4 for undamaged concrete to 3.2 for damaged concrete. The shape parameter W^{*} decreases from 2.2 to 1.83.

Any explosive detonation causes blast pressure followed by gas pressure being not so intense but of long duration. Although the combination of several components could be worse than the sum of their separate components, in most cases it is sufficient to treat their damage effects separately. The relevant failure mode after explosions may be determined using a P-I diagram (TM 5-1300). For a duration of $t_d < 0.25T$ it is customary to characterize blast pressure in terms of impulse magnitude and scaled range

$$z = \frac{R}{W^{1/3}} \qquad (6)$$

with Z for the scaled range, R for the radial distance between explosion center and target and W for the explosive weight (equivalent TNT weight). Relevant blast parameters dependent on the scaled distance are shown in Figure 2-10.

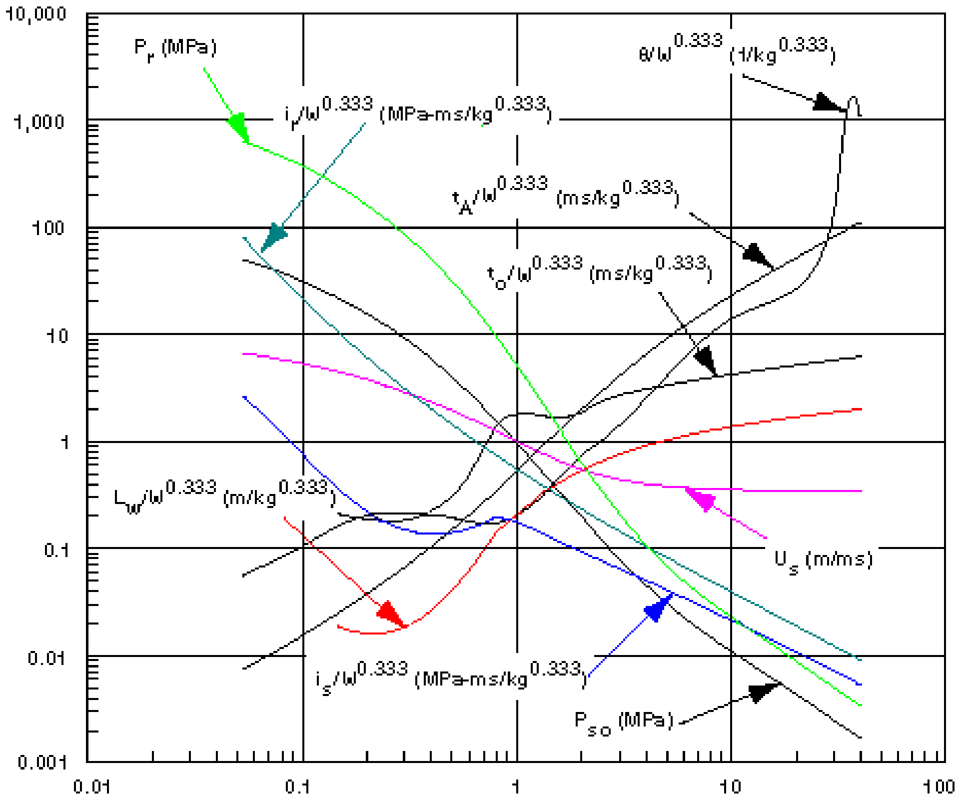

Figure 2-10: Blast Parameters dependent on Scaled Distance
$z = R / W^{0.333} \left[m / kg^{0.333} \right]$ *(TM 5-1300; 1994)*

Except for tunnels or enclosed passages the pressure intensity at a reflecting surface diminishes with increasing distance from the explosion source. At intermediate scaled ranges $z < 1 \left[\frac{m}{kg^{1/3}} \right]$ both global plastic deformation from blast and local damage near the point of munitions impact are generated (Amde, M.; 1996). Charges detonated at very short stand-off distances, i.e. with a scaled distance of $z < 0.2 \left[\frac{m}{kg^{1/3}} \right]$, cause an impulsive air wave and severe local damage taking one of the four forms (see also Table 2-1 and 2-2) :

Penetration (displacement of a missile into the target), scabbing (ejection of material from the rear face of the target), spalling (ejection of material from the front face of the target), or perforation (complete penetration through the target - with or without residual velocity). Additionally to spalling and the more dangerous scabbing, fragments might be propelled as missiles leading sufficiently disrupted buildings to collapse. Disintegration near inadequate connections often is more dangerous than deflection of an element incapable of absorbing the imparted kinetic energy. The damage potential depends on deformation resistance, projectile properties, fragment response, fragment fixty to surrounding structural parts and on loads transferred from other components.

The effect of reflection from structural elements may also be considered with (Biggs, J.M.; 1964).

$$\frac{\Delta P_r}{\Delta P_{so}} = R_L = 2\frac{(7P_A + \Delta P_{so} * 4)}{(7P_A + \Delta P_{so})} \qquad (7)$$

For low and big overpressures the relevant ratios R_L are 2 and 8, respectively. Another proposal is directly relating $P_{r,max}$ to the scaled distance (Low, H.Y. / Hao, H.; 1999)

$$P_{r,max} = \frac{60.388}{z} + \frac{1389.7}{z^2} + \frac{314.47}{z^3} \qquad [kg\ TNT/m^{1/3}] \qquad (8).$$

Type of damage	P_{so} **[KN / m²]**
Window glass breakage	*1.0-1.5*
Debris and missile damage	*1.5-2.5*
Personnel knocked down	*7.0-10.0*
Failure of wooden houses	*7.5-15.0*
Serious damage of reinforced concrete buildings	*40.0-60.0*

Table 2-1: Damage Type dependent on P_{so} for Blast Side-On Overpressure

Reference blast point (m)	*Scaled Distance* $R*W^{-1/3}$ *[m/kg*$^{-1/3}$*]*	*Peak incident pressure* P_{so} $[KN/m^2]$	*Peak reflected pressure* P_r $[KN/m^2]$
10	*2.3*	*276*	*1034*
20	*4.6*	*52*	*144.8*
40	*9.1*	*22*	*34.47*
60	*13.6*	*10*	*20.68*
80	*18.2*	*7*	*15.14*
100	*22.7*	*3*	*10.35*
150	*34.1*	*2*	*5,52*
200	*45.4*	*1,72*	*3,79*
250	*56.8*	*0,83*	*2,07*
300	*68.1*	*0.55*	*1.38*
350	*80*	*0,34*	*1,03*

Table 2-2: Example of the Relevant Values for an Equivalent Load of 85 kg TNT (Amjad, M. A.; 1993 / TM5-1300):

The positive phases of most transmitted shock waves are extremely short leaving nearly no time to respond before the negative phase begins.

$$t_{crit} = 0.25T\sqrt{\frac{R_e}{P_{max}}} \qquad (9)$$

The critical time within which a blast wave impulse must be received by a target for local damage to be inflicted is a function of the maximum pressure P_{max}, lateral local resistance R_e and of the natural period T. For free vibration the critical time $t_{crit} = 0.25T$ (afterwards only overpressure and not duration is relevant (Kinney, G.F. et altri; 1988)). In most cases a triangular pressure-time relation is sufficient. However, if not only the actual stability is relevant, but the loading dynamics themselves (loads largely vary in time or space), also the shape of the load distribution is important. For nuclear weapons with negligible blast duration and pressure decay, one may assume a specified static peak overpressure superimposing dynamic or blast pressure. For conventional weapons, charge duration has to be considered

$$F = \int_0^{t_0} \Delta p dt \qquad (10)$$

using the integral of the impulse or the pressure function over time. The exponentially decaying function

$$P(t) = P_{r,max} e^{\alpha} \quad (11)$$

contains $P_{r,max}$, t and e^{α} representing the peak reflected pressure, positive phase duration (if the negative phase is to be ignored) and pressure rate decay.

Now, the structural response may be calculated using the equation of motion

$$M \frac{d\,x^2}{d\,t^2} \textit{(inertia effect)} + R(x) \textit{(lateral resistance)} = AP(t) \qquad (12).$$

Elastic deformations x_e do not weaken material and primarily depend on the most uncertain value T (the eigenfrequency may be calculated with $\omega \cong \frac{10}{n}$ with n for the number of stories). So, with the mass M and the maximum lateral resistance R_e as well for the dynamic case a conservative description is

$$x_e = \frac{R_e T^2}{4\pi^2 M} \qquad (13)$$

The short-term resistance is considered to be 140% of the long-term resistance. Normally the fact that $P(t)$ varies with time, while the resisting force $R(x)$ with displacement, would require a double numeric integration. However, as the inelastic range of the resistance-deformation curve $R(x)$ is similar to the resistance-time curve $R(t)$, resistance may be expressed as a function of time rather than of distance (Kinney, G.F. et altri; 1988).

The major thrust of fracture research concentrates on deterministic mechanics neglecting uncertainties related e.g. to crack position/size/number. Therefore some 50 years ago Freudenthal established the field of structural safety and reliability, e.g. applied for fatigue prone structures such as bridges. Here, the required fatigue crack growth data for the analyses may be obtained e.g. via MC simulation producing non-stationary random processes representing an important part within computational analyses (Schueller, G.I.; 1997).

At present damage assessment is performed using replica scale modeling, lumped mass/spring mechanical analogues, empirical formulae, FEM, FDM or Path-integral methods. Most computer codes dispose of a graphical interface guiding the user in selecting the best solver for each portion of the problem and in conveniently switching to another solver when this becomes most efficient and appropriate. Deterministic analyses use limit values for local and global stress, deformation or strain, while stochastic analyses additionally consider random or spatial correlated uncertainties. A numerical procedure should carefully be selected. Important factors are the type of material, structural configuration (number of DOF), environmental and loading conditions (deterministic /

stochastic / linear / nonlinear / static / dynamic), damage level and failure modes. Besides, one must consider type and spatial resolution of the sensors, degree of measurement noise pollution, complexity of the detection scheme, nature of instrumentation network and available computing resources.

For nonlinear problems the effectiveness of an algorithm to iteratively solve depends on the rate of convergence. Whereas semi-empirical methods apply simplified models, first-principle methods are based on solid physical grounds. When analyzing explosion effects often the conventional stress-strain relationships break down. Computational Solid Mechanics use mesh-based techniques whose grid[4] deforms and moves with the material thus limiting numerical diffusion, while Computational Fluid Dynamics use global meshless methods whose grid[5] is fixed in space, i.e. the material flows through it. In order to benefit from the power of both techniques and to a less degree inherit some of the weakness of each, CFD and CSM are coupled. Smooth Particle Hydrodynamics and the Element-Free Galerkin method use an approximation function based on moving least-squares and flexible nodes to adequately represent crack propagation and fluid/structure interaction considering both, fragment size and velocity distribution (Sauer, M. 1998). Euler formulations refer to the deformed condition to adequately describe detonating gases, expanding air or liquids (having no shear strength, they are not subjected to mesh-tangling or -distortion). Lagrange formulations describe the undeformed condition including the time history of solids or projectile penetration. Combining an Eularian and Lagrangian grid is a difficult task generally being performed by specialists.

AUTODYN is an engineering analysis program optimally dealing with solid, fluid and gas dynamics including non-linearities, explosions, impact and penetration, contact problems, shock and blast waves (AUTODYN; 1998). It is capable to capture large strains, large deformations and fluid-structure interaction and so, is an explicit code solving the physical equations of mass, momentum and energy conservation coupled with material descriptions. Alternative numerical processors can be selectively used to model different regions of a problem, a multi-physics approach. Lagrange processor for modelling solid continua and structures, and the Euler processors for modelling gases, fluids and the large deformation of solids. Figure 2-11 illustrates their application in Autodyn (AUTODYN; 1998 and Rötzer, J.; 1997).

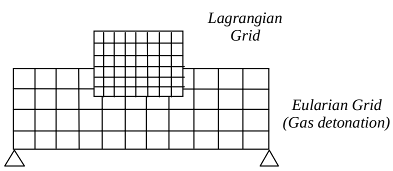

Figure 2-11: Combination of an Eularian and Lagrangian Grid

[4] explicit administration of the topological relationship between mesh nodes is very time-consuming
[5] in contrast to its generation its filling with various shaped structures can be very difficult

When modeling blast one should consider the application period and duration, but also the velocity, peak intensity/amplitude or attenuation of the exerted pressure. As concrete and explosion gas have a different density and velocity, the relationship between contact force and relative displacement is hard to quantify. Extensive data of case studies help compensate existent deficiencies and gain a better understanding of structural behaviour. Quasi-static analyses magnifying static loads are insufficient, since after an explosion, material-independent relations in form of mass (continuity)-, impulse (Navier-Stokes, Euler)- and energy conservation may only serve for a rough estimate, since high pressures change material density and internal energy. When modeling impact one should illustrate geometry (size, thickness), stiffness (mass, eigenfrequency) and relative strength of both, projectile and target. For low impact velocities this is governed by the strength of the target, but insensitive to that of the projectile whose erosion reduces penetration performance (Gold, V.M.; 1996).

The deformation of the target (wall or slab) will be elastic coupled with rebound forces, while with increased missile velocity the target reaction will be in the plastic range characterized by spalling (Perry, S.H.; 2001). At high missile velocities first, scabbing will occur combined with shear plug formation finally followed (with further increased velocity) by perforation. However, in order to produce damage two requirements have to be satisfied, namely the achievement of a critical peak contact force and a provision of minimum impact energy. Experimental evidence suggests, that the primary phase of shear plug formation depends on missile velocity and mass, target thickness and damage extent incurred by the structure. The secondary phase is governed by the structural response and, in case of perforation, by the reinforcement. The maximum force is related to local damage, while the impulse usually influences overall failure. So, rehabilitation should aim at an improvement of local shear resistance by using adequate shear reinforcement producing overall bending.

Dealing with fluids and turbulence in the air as a response to blast is just as difficult as dealing with a lack of constitutive equations for failing solids after fragmentation. Viscosity of fluids and fracture toughness of solids play the same role in the physical similitude description of continua separation and, therefore can be studied by the same criterion of instability (Carpinteri, A.; 1983). Experiments show that structures under uniformly distributed impulsive loads or under repeated cycling loading predominately fail locally by shear at the supports or at other critical locations in a plane parallel to the loading direction (vertical slip) with few deformation or, through bond (Krauthammer, T; 1993). This fact shows even stronger evidence in case of high stiffness. Punching cracks may even occur at the beam midspan and immediately after load arrival, before a meaningful global flexural response has been developed. At high impact speeds or short loading duration shear dominates and so may be uncoupled from flexure representing a lower vibration mode with longer loading duration. However, dynamic loading produces diagonal or direct shear and moment interaction, both causing a section to fail at lower than the respective ultimate strengths. Here diffuse flexure in the hinge region might have begun along with the continuous development of in-plane tension (Menetrey, Ph; 1997, Krauthammer, T; 1993). Therefore, constitutive modeling in transition regions from predominately flexural behavior into domains dominated by punching shear still must be optimized.

Nonlinear behaviour may be detected either directly by looking for nonlinear characteristics or indirectly by looking for violated assumptions of a linear system (physically not interpretable values such as negative mass). Structures often are linear in the global sense, even when there are local nonlinearities. They may be tailored by a computer program (e.g. Adina) treating some regions as linear (e.g. floor), while other areas are treated as nonlinear. Provided that extreme caution is used in conducting the analyses and in interpreting the results, in this way areas of stress concentration or inelastic deformation may be detected. Structures subjected to non-oscilliatory loads such as blast may be dealt with by using single degree of freedom simplification if their deflected shape is equivalent to that resulting from the static application of the dynamic load (Biggs, J.M; 1964). Uncertainties in loads, material and support characteristics (torsion coupling) generally do not warrant the effort of complex analyses with exact values for inertia, stiffness and flexibility. Soil-structure interaction should be described with frequency-independent impedance[6] functions to consider propagation towards infinity (depth extrapolations made without impedance functions could exhibit growing oscillations, much like a physical system receiving energy from an external source).

For negligible stored kinetic energy the structural response is linear elastic (not the case for high impact speeds). Single degree of freedom spring-mass models with a console scheme of linear springs and masses lumped at the floor levels, modeling inertia forces in combination with partial systems such as beams, plates or shells is appropriate (Kinney, G. F. et altri; 1988, Bangash, M.Y.H.; 1993 and Jones, N.; 1989). The repeated application of equivalent static forces at probable hinge locations or of a virtual deflected shape corresponding to the natural eigenperiod may help calculate the effect of dynamic strain rate effects or system degradation using a fatigue factor of $\mu = 3$, provided that the dynamic part is below *10%* of the static part (Chun-Man, C. et altri; 2000). For impulsive loading the curves of Figure 2-12 may be applied:

[6] special class of mathematical functions that describe causal, linear disturbances in physical objects dissipating energy (e.g. depth extrapolation of waves); they play a fundamental role in any modeling calculation where time evolves from past to future. By formulating extrapolation problems (unstable solutions) with impedance functions, stability can be ensured which is not always possible by “straightforward” implementations of physical equations.

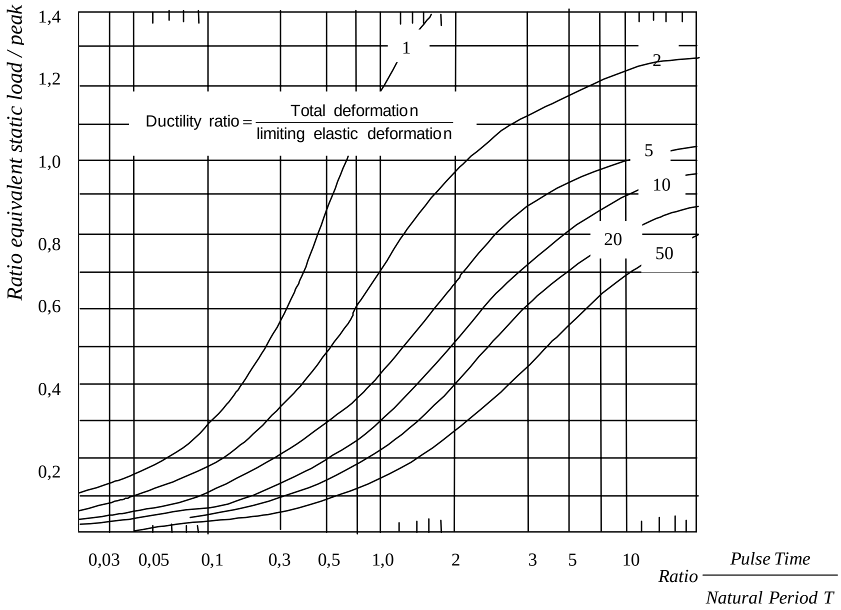

Figure 2-12: Ratio of the Equivalent Static Load (Function of Ductility Ratio, Pulse Time and Natural Period T, Kinney, G. F. et altri; 1988, SIA; 1996)

In a push-over analysis acceleration time histories of gradually increasing equivalent static loads of e.g. *10%* of the ultimate load are imposed until movements have ceased or the crack cause is identified (Okamoto, S; 1990, Kakodkar, A; 1995 Bracci, I. M. et altri; 1997 and Chun-Man, C. et altri; 2000):

- initial elastic stage (moments below the yield capacity are harmless)
- first yield of one, in large structures of several members (stiffness change leads to a new vertical distribution of the lateral loads)
- incipient mechanism (plastic hinge formation due to yielding)
- final failure mechanism (site-specific demand curve intersects the force-deformation capacity -limit stress and limit deformation)

In addition to global stiffness before and after loading, it is important to know, *when* the global or interstory drift exceeds a specified limit and, the amount of dissipated energy in each of the four different stages of structural response. A structure is considered as safe for a drift below *3-4% H* (*2%* of the building height *H* if brittle), while unserviceability already begins at *0,5% H*. Note, that soil condition and eigenfrequency significantly influence structural resistance against lateral loads.

The stiffness reduction due to lateral drift may be captured with less computational time by directly considering the effect from load induced cracks. This avoids the cumbersome activity of gradually imposing incrementally loads as is done in push-over analyses (Fling, R, S; 1997 and Chun-Man, C.; 2000). Such incremental load analyses may adequately describe serviceability up to *70%* of the ultimate load. Then, the induced lateral deflection represents only a rough estimate for the change of stiffness, since cracks change already occur at *15%* of the ultimate load. Partially cracked beams suffer a large deflection with a stiffness less than a third of the flexural stiffness $EI_{uncrack}$ for uncracked cross-sections (Fling, P.S.; 1997)[7]. Responsible here is the moment of inertia *I* and not the modulus of elasticity *E*. The application of the "Direct-Effective Stiffness Analysis" with $I \approx h^3$ for uncracked cross-sections, $I \approx d^2$ for cracked sections and $I \approx d$ for heavily reinforced sections has been verified in numerous studies. A reduction of the moment of inertia I_{uncr} by *80%* for columns and by *50%* for beams adequately represents reality at least for a screening evaluation. The cracking moment approximately equals

$$M_{cr} = \left(\sigma_v + 0.6\sqrt{\beta_D}\right)\frac{I_{uncr}}{y_r} \qquad (14)$$

with σ_v for the axial compressive stress, β_D for the compressive strength of concrete and y_r for the distance from centroid of the cross-section to its extreme fiber in tension (Chun-Man, C.; 2000). A change in stiffness also can be indirectly determined via the eigenfrequency (first usually is sufficient) with:

$$\omega^2 = \frac{EI}{M} \quad and \quad \omega = \frac{\pi^2}{l^2}\sqrt{\frac{EI}{\theta A}} \qquad (15).$$

In case of static loading and small displacements material parameters may be considered to be constant, as there is a linear relationship between external loads and internal stresses. The frequency content and mode shapes are irrelevant, and different relative stiffness only changes the sequence of plastic hinge formation. Dynamic forces, however, affect the material even changing if stiffness values are over- or underestimated by the same factor. Here, the yielding sequence of the members and potential soft-story mechanisms are of crucial importance. Stiffness singularities (10% represent damage) or changes in the frequency curvature inform about general strength deterioration (Langhe, K.D; 1997). For all eigenfrequencies of interest local parameters such as material anisotropy stress/strain distributions may be deduced from velocity and acceleration integration. Subjected to many uncertainties these only allow a rough

[7] A more exact estimation of the moment of inertia I for cracked sections may be performed using Figure 4-11 and 4-12 of the manual TM5 - 1300

estimation of structural performance and must be complemented by an additional analysis of local damage indices. Experience has shown that even severe damage may not become obvious if the structure is highly redundant (Hogue, T.D. et altri; 1991, Salvaneschi, P. et altri; 1996).

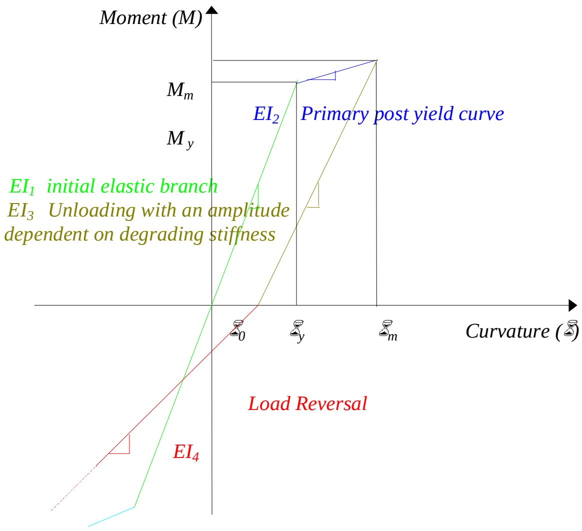

Figure 2-13: Moment - Curvature Relationship (Selna, L.G. et altri; 1998)

For buildings with more than 5 stories detailed real-time analyses with a variable number of DOF should replace simplified analyses based on quasi-nonlinearities such as damping and secant stiffness. In a combined dynamic-static FE model the dynamic part may e.g. describe the global structure including its number of DOF and relevant modal parameters such as amplitude, frequency and shape of excitation, temporal variation of eigenfrequency, and geometrical, material or viscous damping (see Figure 2-13 and Langhe, K.D.; 1997). The burdensome re-computation of eigenvectors for each time-step with proceeding time-history now can be replaced by reduction techniques reusing stored eigenvalues as far as possible (see also Selna, J. and Kim, K. 1998). With the mass supposed to be unchanged and nonlinear inelastic behaviour only occurring in hinge regions, tangent stiffness *EI* equals the quotient of the bending moment to the corresponding modal curvature. The radius of the curvature is a variable describing local rather than global behaviour.

Also with unavoidable simplifications a model may only represent real structural behaviour if reasonable approximations for the support conditions are selected. The restraint between a free and a fixed rotation depends on the relative stiffness of e.g. walls to slabs. As a rigid fixture practically cannot be realized, it is much better to define an elastic boundary and then, at the global-local interface model adapt the results for the required conditions. Experiments should test asymmetry influence, contribution of internal columns, diaphragms and bearing walls against horizontal actions, floor capacity for effectively bracing and, the behaviour of the joints between corners of facade walls or between slab and wall. If the latter are not reliable or difficult to quantify, one had better not to include their structural function in the model. When determining maximum eigenvalues, the influence of damping on torsional stiffness can be neglected.

Soil vibrations due to blast e.g. may be evaluated like seismic excitations by using an equivalent linear system. Dependent on soil stiffness and flexibility, equilibrium and compatibility equations can be adjusted at the end of each iteration until convergence is achieved. Bounding soil cases should account first for soil and then for structural uncertainties. Equivalent strain-compatible soil properties are obtained by curves correlating the expected shear strain level versus shear moduli and soil damping. In order to prevent wave reflection due to a lack of system damping, an artificially viscosity removed after the wave has propagated should be introduced at the boundaries (Humar, J. L. et altri; 1998). A fixed base condition may be assumed, if the shear wave velocity exceeds 1100 m/s with a corresponding shear strain below 0.01.

Computational analyses help decide about a systematic and cost effective approach for repair or replacement. Finite modeling is indeed the best representation for a complex interaction of structural elements. However, the custom detailing of each joint, element and spring to match a real structure is an art-science which is quite cumbersome, time consuming and prone to significant errors. Deterministic methods only working with limit values, such as difference between real and estimated parameter or number of times an event occurs, should not only be combined with probabilistic methods, but also with neuro-fuzzy techniques quantifying the extent of reliability with respect to an opinion or to a value. In most cases, an isolated application of probabilistic methods has been unsuccessful in damage assessment always suffering from sparse information (Hartmann et altri; 1997 and Schnellenbach-Held, M; 2000). Not only that a selection of type and location of samples or sensors may be inadequate, but geometry, loading, or material conditions are also subjected to numerous random and nonrandom errors. Actually, an unequivocal measurement of the phenomena of interest often being noisy does not exist. Besides, failure has often been caused by misuse and over-tolerance on the accuracy of computed solutions not necessarily being correct just as the precision of computational results is high. Again, computers were intended to explore a wide range of alternatives and to relieve the engineer of repetitive tasks allowing more time for quality control. However, in many cases the right questions are not even being asked.

2.3 Geometry Recording and Material Investigation

The rapid progress in computer technology with powerful, refined analysis methods has cast some doubt on the value of experimental analysis. Eventually, on-site inspection and carefully monitored physical experiments or laboratory tests are indispensable for the interpretation and calibration of computed solutions. Besides, geometry recording and material investigation are indispensable to obtain an exact picture of the real structure necessary to begin with computational analyses. The geometry and position of decisive bearing elements, deformation or story drift may well be determined using tachymetry or photogrammetry. However, overall dimensions, structural configuration and cross-section properties may also be determined with traditional methods such as topography or with GPS (satellite), laser geodesy, telescopes, binoculars, optical micrometers, telecoordinometers, 2D/3D scanning or laser distometers (see Figure 2-14).

Figure 2-14: Laser - Distometer

The latter inform about distances or elevation angles and relate them to on-line calculated coordinates (Altan, M. O. et altri; 1998). Asestometers, electric piezometers, extensiometers, deflectometers, deformameters and strain gages together with glued reference marks help define damage induced displacements. Visual examination only allows a preliminary identification, if the vicinity of damage is known a priori and, if the relevant variables are observable such as deflection, crack position/width/length, concrete cover, humidity, pH value, content of chlorine in the air and carbon dioxide. Unobservable variables such as shrinkage cracking, corrosion state, creep, relaxation or freeze-thaw cycle attack are recorded using different tools.

A very high resolution (insensitive to temperature or humidity influences) at high costs may be obtained by the application of a light-wave-conductor which conducts light differently dependent on the damping characteristics of material. Holographic interferometry[8] is based on the interaction of two or more waves producing interference, that is a new wave with twice the amplitude and four times the energy of a single wave. Here, their beams e.g. are superimposed in a way that the crest of one wave will add up to the crest of the other, provided that the arrival times are the same (Daniel, I. M.; 1983 and Krieger, J; 1995). By deducing stress from strain distributions, in this way the material response (crack initiation and propagation) may be characterized. The moiré holography combines features of several methods and modifies the optical components to achieve a better form for processing.

[8] ínterference is an interaction of two waves, whose size (amplitude) and degree to which they are in or out of step with each other (phase), is decisive whether they will add together (e.g. in case of resonance) or cancel

The Electronic Speckle Pattern Interferometry uses monochromatic light of laser beams refracted through a backward wave containing the same intensity (amplitude) and phase characteristics (hologram) as those of the original object. So, object and reference beams impinging on a film plate allow reconstructing the flaw shape via optical interference. In order to obtain an interferogram the speckle effect (deformed and undeformed surface) is registrated digitally. In contrast to usual photographic methods, it provides as well spatial information of a body, provided that it is illuminated with several colours each producing an own picture to avoid confusion. The moiré photography is based on contour mapping and compares gratings in original and distorted condition. The quality of its results depends on the deformation magnitude always being impaired by local imperfections such as lack of planeness. Whereas coarse gratings use mechanical interference with geometrical optics (rectangular light propagation), high frequency gratings apply optical diffraction or interference of coherent light components. Before any measurement takes place, the specimen surface has to be prepared with a smooth and partially reflective substrate. Not only the probe spacing, but also the surface contact should be optimal.

Impact Echo tests are performed to assess the conditions of slabs, beams, columns, walls (see Figure 2-15). In general, the hammer is used to generate compression waves reflecting back from the bottom of the tested member or from a discontinuity. System response is measured by an accelerometer receiver placed next to the impact point. Both, the hammer input and the receiver output are recorded by a digital analyzer. The time traces are then transformed to the frequency domain for calculations of the transfer and coherence functions, and the auto power spectrum of the receiver. One of the advantages of the IE method over the Ultrasonic Pulse Velocity method is that only one side of the structure needs to be accessible for testing:

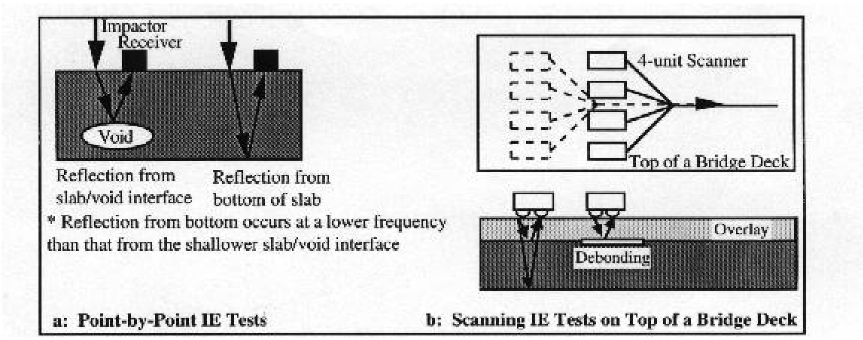

Figure 2-15: Impact Echo Test Configuration (http://www.olsonengineering.com/ie)

A low electrical resistance *(<200 KOhm)* informs about a lack of protection indicating reduced effectiveness of the concrete cover or a failed coating. Acid titration indicates exposure, i.e. infiltration of aggressive agents including chloride content (safe limit against corrosion 0.2% of the cement mass) and sulphide content (safe limit 4% of the cement mass). The carbonation depth (alkalinity) may be determined with the help of an acid based phenolphtalein indicator turning into purple red at surfaces where the pH-value exceeds 10. This is the case e.g. after fire, where disassociation of $Ca(OH)_2$ already begins at 470°. In the analyses heat transfer through concrete must be determined before a temperature rise in steel which practically achieves complete recovery. After cooling concrete suffers further loss in compression strength (Nassif, A.; 1999). Heating (tension) and cooling (compression) produce thermal infrared radiation.

Infrared thermography may assess future durability of fire damaged components with minimal disruption and without mounting strain gages. Here, thermograms contain images in different shades of colour to indicate the change in temperature a component has been exposed to. Unfortunately, they do not show crack depth or distribution along thickness (penetration depth only a few cm), require complex schemes for signal processing and are difficult to interpret, especially if different kinds of defects occur simultaneously. Although calibration improves the situation, field applications are rare (Krieger, J.; 1995).

The half-cell potential informs about active deterioration and delamination processes in e.g. expansion joints (see Figure 2-16). The more negative it is, the greater the corrosion probability. It may indicate the location of rebar corrosion, but not its rate. In areas, where the digital voltmeter measures a difference in potential below *-250mV*, supplementary tests are required to establish the extent to which the steel is corroded (Concrete Society; 2000). For a corrosion probability higher than *90%*, experts recommend to additionally determine cover depth, carbonation depth, chloride content of the concrete, concrete resistivity and exposure of reinforcement. Especially in case of high humidity, corrosion induced tensile forces may lead to section loss. Equi-potential lines indicate anodic (high corrosion risk) and cathodic (low corrosion risk) areas:

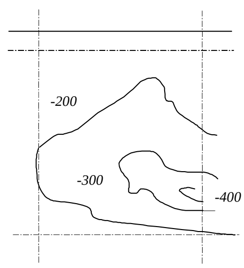

Figure 2-16: Half-Cell Potential Map (Concrete Society; 2000)

Errors are minimized, if the measurement equipment is maintained well, the environmental conditions are adequately taken into account, and the surface area above the suspected corrosion prepared to rule (Concrete Society; 2000).

A scanning electron microscope predominately illustrates cavities, vaults, flaws, inclusions (micro-structural imperfections), loose connections and crack geometry. In the same way also magnetic inspection tools, Infrared Thermography or Impulse Echo indicate areas of stress concentrations and defects. The same applies for the Ultra Sonic Velocity informing about areas without concrete. Its tests are performed to assess the conditions of structural members with two-sided access such as beams, columns, walls, and elevated slabs. Voids, honeycomb, cracks, delaminations and other damage in concrete, wood, stone and masonry materials can be located with this method. Such discontinuities force a sonic impulse to detour around them dependent on refraction thus increasing the path length of transit time. Highest extension of cracks corresponds to lowest pulse velocities.

Based on time measurement of an electro-magnetic impulse (e.g. of a Georadar) Impulse-Radar tools are emitted by an antenna and reflected completely or only in part, dependent on the characteristics of the relevant material. The distance of the reflecting surface to the antenna may be calculated with (Maierhofer, C. et altri; 2000 and Krieger, J; 1995)

$$d = \frac{\Delta tc}{2\sqrt{\varepsilon}} \qquad (16)$$

Here, ε represents material dielectricity, $c = f \times \lambda = 3 \times 10^8\, m/s$ light velocity, Δt time interval of reflection between front and rear face. Since ε for reinforcement is quite high (good reflection), this may be localized very well. A determination of size, location, depth, direction and condition of embedded reinforcement or tendons with a magnetometer or covermeter is possible, if there are no disturbing metals nearby. Its high-frequency waves propagate through the material and reflect wherever there is a change in the dielectric properties disrupting the magnetic field. An existing mesh reinforcement disturbs accuracy and therefore often represents a problem.

Micro-Power-Impulse Radars, especially developed for bridge deck inspection, are similar to other ultra-wide-band radar tools, but are more compact and inexpensive to manufacture (radiate at shorter pulses). Better than the above mentioned techniques Infrared Thermography can show heat insulation or areas subjected to tension (temperature rise) and compression (temperature decrease). Originally developed for the study of local star systems contact-less laser Optical Interferometry may well illustrate whole surfaces and small angles or tiny distance increments. Whereas intensity investigations use vector quantities of the vibro-acoustic field measuring local energy transport, Radiography - determination of diameter and loaction of rebars- applies γ rays to produce a signal (Krieger, J.; 1995, Süß, H.; 1999).

The mechanisms of stress or strength characteristics usually are determined with a rebound or impact hammer (see Figure 2-17). The surface of the concrete is cleaned to remove shutter board marks. A series of twelve readings are taken with a 'Schmidt' hammer with a drop weight of ca. 6 kg in a diamond pattern at the test point. The hammer is pressed against the surface, loading the sprung mass, and releasing this at the end of the stroke producing vibrations in the low frequency range. The percentage rebound is measured by a latched rider on the side of the hammer. The conversion of rebound number to compressive strength can be achieved by preparation of a calibration chart for the concrete concerned. If this is not possible, a crude assessment can be made from the manufacturers data. In practice the test is very dependent upon the surface condition and moisture content of the concrete as well as the ratio of aggregate to cement paste. Since low frequency waves only allow a good resolution if the material is highly damaged, they should be restricted to local flaw detection for small structures. In contrast, an impulse force generally is in the higher frequency range ($f < 250Hz$, Javor, T. 1991).

Figure 2-17: All James Test (ASTM C805, Korea) and Schmitt Hammer

In order to exploit the utmost information from the signals, these first have to be adequately recorded, processed, filtered, visualized and finally interpreted (see chapter 3.2). Due to the unavoidable limitations of all Non Destructive Evaluation methods, detailed information about the extent of damage can only be obtained by combining several techniques. In this way the condition of a structure can comprehensively be illustrated (Krieger, J.; 1995). In areas exhibiting extensive deterioration (rust spots, cracks parallel to the rebars or breaking away of concrete on the top of the rebars) or, where quantitative results are desired, these should be complemented by destructive coupon sampling (core drills not exceeding 12.5mm in diameter may be considered to be nondestructive). Variations are inevitable and should be calibrated by additional core testing also to be used for chemical or petrographic testing.

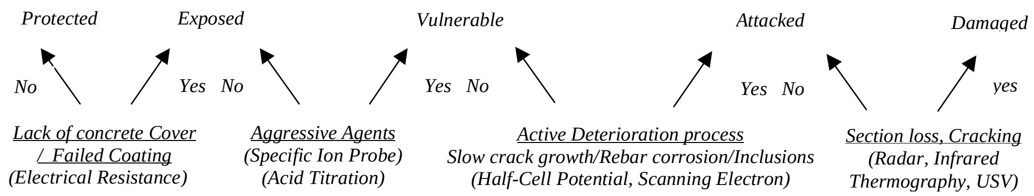

Figure 2-18: Environmental Condition Stages of RC Component (Hearn, G. et altri; 1998)

Figure 2-18 shows chemical or mechanical degradation (abrasion, water penetration, weathering or solar radiation), biological attack (funghi or moisture), aging and thermal forces due to -by fixings constrained- displacements or stiffening members may endanger a structure in the same way as static overloading, cyclic loading, impact, ground movement, settlement or nearby excavation. Investigation to diagnose corrosion (in concrete) includes a determination of the cement and Cl^- content, neutralization measurement, quality of residual strength survey and resistivity measurement (in steel). Corrosion rate, concrete cover and a possible reduction in rebar diameter can be illustrated with potential contours. Service life is a progression of condition stages over time whose assessment requires experience, statistical sampling theory and physical measurements. NDE qualitatively informs about size and amount of microscopic and macroscopic flaws classifying a structural component into “protected, exposed, vulnerable, attacked, damaged” by comparing disparate threats (Hearn, G. et altri; 1998). The Markov Chain, special type of stochastic simulation, determines transition probability between several condition stages (present probability distribution only depends on that one at a previous time).

If the structure is readily accessible, additional in situ and/or localized laboratory investigations should strengthen or disapprove the original hypotheses at least for some critical areas (see Table 2-3). A petrographic examination of type, size, homogenity and distribution of concrete aggregates and cement content shows if alkali-silica reaction is probable. Poston proposed a new method, namely the SHRP test (Poston, P.W; 1997). Water absorption capability represents the volume absorbed into a concrete surface from a standard reservoir (< 1cm in 4 minutes) indicating air permeability (porosity). Fatigue in steel elements should be assigned on the basis of exposure to cyclic stress and degree of exhaustion. Break-off, pull out tests or cored real samples for laboratory investigations inform better about strength or bond characteristics than field tests with natural system’ input. Nevertheless they should be minimized as far as possible in order to maintain structural entity. Only if they are carefully prepared, representative results may be achieved and, expensive but useless operations avoided.

Visual Inspection, Expert Knowledge and Intuition

↓

In Situ and Laboratory testing	
Mechanisms of Distres *1. Conrete cover/ Coating failed (Electrical Resistnce)* *2. Aggressive agents -Chloride-, Sulphide Ion and Oxigen content* *3. Acid rain (Specific Ion Probe, Acid Titration)* *4. Carbonation depth (Phenolphtalein indicator/PH-Test)* *5. Alkali-silica reaction of cement and aggregate dependent on the cement content (petrographic examination) –physicochemical disintegration* *6. Rebar Corrosion (Half-Cell Potential) -electrochemical deterioraton* *7. Section loss, Cracking (Radar, USV, Infrared Thermography)* *8. Stress history, stages of ageing, previous yielding, inclusions (Scanning electron microscope)*	*Strength Characteristics* *1. Hammer Readings* *2. USV* *3. Core Strength* *4. Rebar Capacity* *5. Pull-Out Test*

Table 2-3: Techniques for Condition Assessment

If direct observation of the fault zone is impossible, e.g. closed or access hindered by casing, vibration based monitoring using modal parameters instead of stress may be the only source of information (Hermández, J. M.; 1997, Castillo, E. et altri; 1997 and Wu, X. et altri; 1992). White noise excitation is homogeneous or stationary (Gaussian) with a standard deviation correspondent to that one of the scattering data, i.e. of measurement noise. Colored excitation, however, is subjected to stochastic correlation in time with several independent time arguments (Schueller; G.I.; 1999). In passive diagnosis with the help of e.g. acoustic emission remote sensors register movements during the normal operation of a structure (ambient natural harmonic excitation). USV, however, produces stress waves using active piezoelectricity to evaluate strength, density, hardness and dynamic elastic modulus. Here, hydraulic or electromagnetic actuators or hybrids are required to produce artificial excitation with modes adjacent to the buildings eigenfrequency ω_0 which, virtually is a cumbersome activity. The response waves are directly registered by -on specimen surfaces attached- sensors combined with transducers and linear or nonlinear conversion parts (see Figure 2-19). Modeling errors and their effect on damage assessment may even not be reduced with a unique filter for each sensor. So, methods capable of dealing with nonrandom errors are urgently needed (see chapter 4).

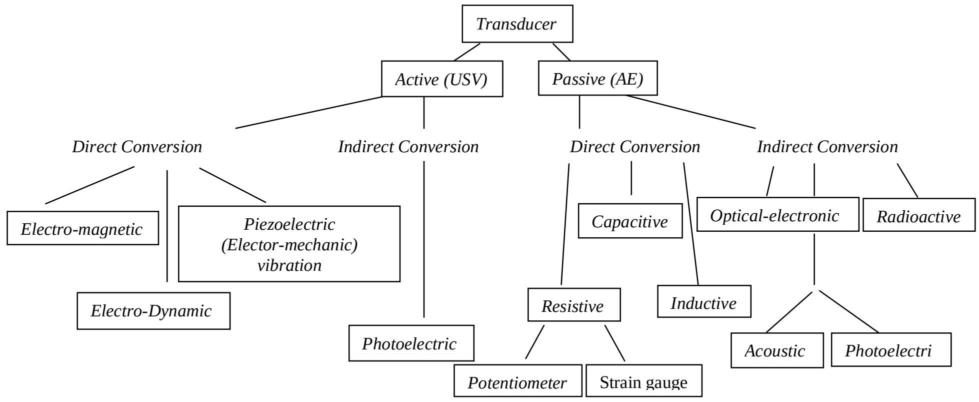

Figure 2-19: Type of Transducer and Data Conversion (Natke, H. G. and Cempel, C.; 1997)

Since time-domain signatures from response accelerations are very extensive, generally they are converted into frequency domain signatures (amplitude, form and attenuation of mode shapes, eigenfrequency and damping) to obtain internal forces and deflections. The transfer functions from accelerometers positioned on an attached steel plate at different locations then have to be compared by calculating the product of amplitude and frequency together with the velocity. Here, a Fast Fourrier Transformation algorithm is used to approximate energy dissipation. Whereas crack location (usually different to that of the sensors measuring symptoms) is determined via reflection, its depth and geometry is deduced from the arrival time of emitted elastic waves (deterioration - increase, Berra M. et altri, 1992). One may e.g. estimate the depth of foundations by placing sensors at different heights on the same wall, since the application of an impelling force on surfaces allows receiving a signal on the opposite surface due to wave reflection (Henrich; 2000).

Strain gages are used in most measurement devices including strain sensors, dynanometers or velocity and acceleration sensors (see Figure 2-20). Velocity sensors (producing a system velocity equal to the induced one) and displacement sensors (producing a system displacement equal to the induced one) are adequate for a low frequency range. Acceleration sensors emphasize damping force in an upper frequency range. They directly measure structural vibrations and so, avoid the cumbersome calculation of stiffness and mass. The determined frequency or mode shape will result in the flexibility matrix.

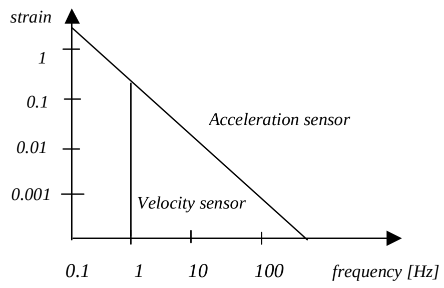

Figure 2-20: Range of Application for a Velocity and Acceleration Sensor

The mutual distance of sensors should comply with system properties, stress peaks, maximum deflection and with the possible type or location of faults (Natke, H. G. / Cempel, C.; 1997). An analytical model does not only serve for simulations requiring data of material properties, but also helps design a testing and instrumentation or data acquisition scheme by determining the bandwidth of expected frequencies in critical areas. Simulation and testing should closely work together. The reconstruction of a modal shape curvature should be as easy as possible using e.g. zero-crossings where forces will not interact with the system. Ideal would be a coincidence with nodes of a mathematical model to avoid interpolation. A measurement grid in conformance with the number of DOF of the system allows optimal spatial resolution and interpretation of the results. However, even calibration may not completely eliminate disturbing effects from electrical interference. So, emphasis should be given to the redundancy of measured modes and inference instruments. The influence of the sampling period (time intervals to which the actuator signal is used), of the record length in the control loop (inherent time delay) and of temperature or water on the response should be verified by additional real-time analyses. The same applies for the frequency response functions prior to any post-processing, since environmental effects, modal truncation or signal conditioning might change eigenfrequency more than damage itself.

How often a time history crosses the initial position is a measure of the predominant response frequency, while the distribution of peaks informs about the number of excursions in the inelastic range. It is difficult, nearly impossible to identify precisely the contribution of a mode, as this depends on the actual damage location being unknown when mode shapes are selected. Failure modes should be scaled[9] in accordance with the model scale adequately satisfying linearity, observability and stationarity to obtain features allowing an accurate and effective correlation with the structural capacity (Aktan, A.E.; 1997). One should select exclusively informative measurands conforming with the wave length and the available energy at that frequency content. Only flaws in the maximum curvature part of a mode shape lead to its sensible influence. Displacement curves over the building height may indicate parts where damage is

[9] The experience obtained for space structures may be applied for civil structures

most probable, provided that they include the entire response between linear elastic and complete collapse (in Figure 2-21 only the members and connections of the 10^{th} to 15^{th} story are critical).

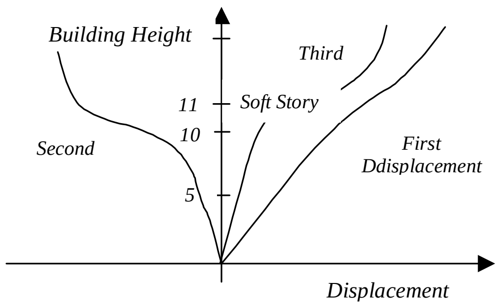

Figure 2-21:Displacement over the Buildings' Height (Filipou, 1998)

Having an increased eigenfrequency and large response amplitudes the upper stories of a building are always more sensitive to lateral stiffness and to resonance. Therefore, an optimal location for a single accelerometer is the top floor. If several are available they should be placed starting at the top and bottom floors, and then progress towards the middle ones. Simply applying them in the middle floors is sufficiently accurate only in case of noise-free recordings (Heredia-Zavoni, E and Esteva, L.; 1998). If it is impossible to attach accelerometers directly on several points along the structural elements' axes, the mode shapes are indirectly calculated from eigenfrequencies. Their localization inaccuracies are only 2 % compared to 15% (mode shapes) and 30% (damping).

Although the displacements are inversely proportional to the 5^{th} power of the mode number, for well capturing the lateral response not only the lowest, but frequencies and associated mode shapes of the first four to five modes, namely 1^{st} bending horizontal, 1^{st} bending vertical, 1^{st} and 2^{nd} torsion should be extracted (see Figure 2-22 and Kinney, G. F. et altri; 1988 and Roca, P. et altri; 1997). The number of necessary modes depends on the damage degree. The heavier it is, the fewer are the modes required to predict it. In general, incrementally load steps are used (push-over analysis) to illustrate the structural response including its complete time-history. Capturing stiffness reduction due to the cracking effect of the ultimate load is possible and requires few computation time (Chun-Man, C.; 2000)

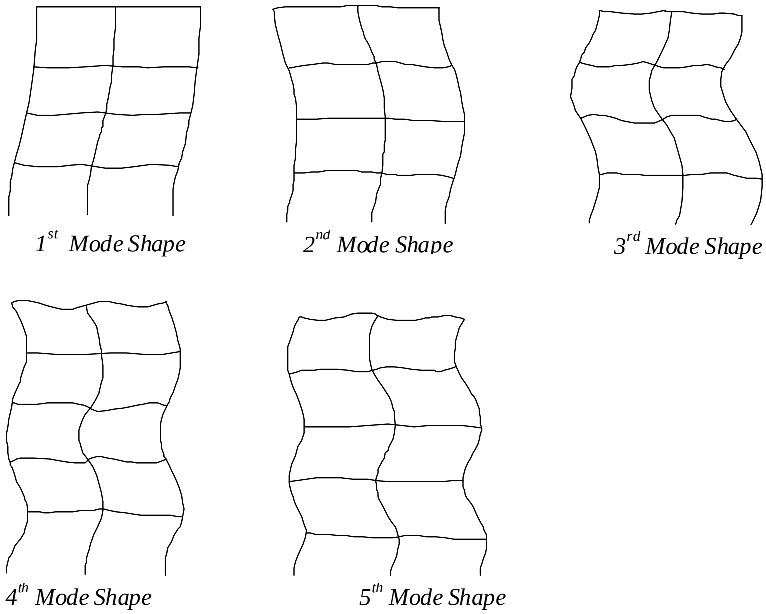

Figure 2-22: Mode Shapes of Multi-Story Building (Selna, L.G. et altri; 1998)

Without an adequate background of type-specific research and an appropriate instrumentation, it is not advisable to conduct simulations or load tests through a structures' entire behaviour range, starting from the undamaged state through its various damage states, and finally to its failure state. As only large defects with magnified signals produce a change in the structural response (experts talk about the loss of a girder) load tests just serve for a global monitoring (Rybicki; 1978). They are very sensitive to changes in the boundary conditions and to temperature. Even with an optimal instrumentation they are neither comparable nor reproducible. Since they only measure stiffness and not strength, highly redundant structures may pass them if their strength is substandard/unsound. So, the loss of pretension or corrosion will in most cases not be detected thus leading to unsafe evaluations. Besides, the high costs of external power sources and an increase in risk of failure during the test requires careful planning and monitoring for risk perception of occupants.

However, as an owner would much rather have a failure during a test than the case when the structure is in service, a limited risk is accepted, provided that one takes special precautions against injury to test personnel. Since ca. 40% of failures already occur during construction, the construction load may serve as a kind of proof load truncating some parts of the resistance distribution (Stewart, M.G.; 1997). In summary, one should aim at a compromise between diagnostic performance low load test and proof-load test which -being 80- 85% of the serviceability load- only detects gross errors on capacity (Moses; 1994).

Serviceability and integrity of structures will certainly remain a domain of NDE, though not allowing a totally unbiassed reconstruction of a flaw. However, also for visual inspection and field or laboratory tests, assumptions have explicitly and implicitly to be made in the algorithms. An optimal signal interpretation requires extensive experience to allow as well a detection of competing effects. System calibration should correct both, random and nonrandom errors of the sensors and of the total measuring chain using several detection methods at the same time. Additional laboratory experiments, static and dynamic simulations, literature and engineering judgement are the only way to strengthen or disapprove the obtained results.

In Vibration testing the following calibration methods are used: In the Static calibration specified sensors equipped with strain gages measure the frequency response for T = 0 while loaded by several known weights. The Direct and Indirect dynamical calibration optically or acoustically measures the dynamic response for a known excitation (amplitude and frequency) by using mechanical or electromagnetic vibrators. The Reciprocal calibration is an absolute method using reference (electromagnetic and piezoelectric) sensors and transducers with linear conversion elements. In the upper frequency range these often are supplemented by reference interferometers sensors (optical calibration). Also, Calibration by comparison is also effective. In order to avoid a calibration for each crack-depth measurement, a self-calibrating USV method has been developed (Daponte, P. et altri; 1998 and Achenbach, J.D. et altri; 1998).

2.4 Ductility and Dissipation Capacity

Ductility and dissipation capacity are among the most important factors enhancing system reliability, for extreme events such as earthquakes the limit state of energy dissipation even make more sense than other values (Ellingwood, B.R.; 1994). They are therefore dealt with in a separate section, since they may very well delay a collapse of a structure. Again, only summarizing the respective value for damage extent is not sufficient. The general vulnerability of a building such as dilapidation degree, ductility, stiffness are in the same way relevant. The capacity of a structure to withstand dynamic stress is a function of both resistance and ductility.

Especially, for earthquakes an energy limit state is more reliable than a strength based limit state. Most critical here is achieving a balance with respect to both, single structural components and the system as a whole. Generally, an increased resistance reduces ductility or energy dissipation capacity, unless specific measures are taken. The smaller the strength, the greater the ductility has to be. Structural behaviour with respect to a possible collapse is approximately equivalent to the product of resistance and ductility (Bachmann, H; 1996). Usually improving ductility is cheaper and -since damage is a consequence of deformation rather than of strength- more effective. If the demand of strength and/or ductility is too high, the structure begins to yield and, redistribution occurs (Figure 2-23).

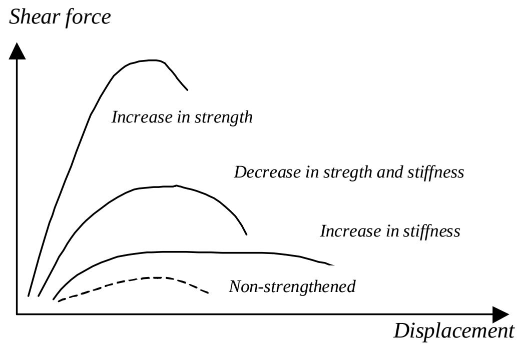

Figure 2-23: Relationship between allowable shear force and displacement (Velkov, M.; 1986)

Since the rotation capacity μ_{rot} of a structural system may ensure safety even if strength limits are exceeded, it is worth to evaluate plastic hinges not only with respect to their location, but also with respect to their characteristics dependent on detailing (see Figure 2-24). It has been shown in experiments that the rotation capacity / rotation ductility of reinforced concrete sections is strongly influenced by the post-yield stress-strain response of reinforcement rather than by the material properties of concrete. It is therefore of crucial importance that additionally to the member ductility (curvature ductility, rotation ductility, displacement ductility) the reinforcement steel *itself* be ductile. In contrast to columns beams can be designed so that their neutral axis remains below half the effective depth d to allow for a strain-hardening response.

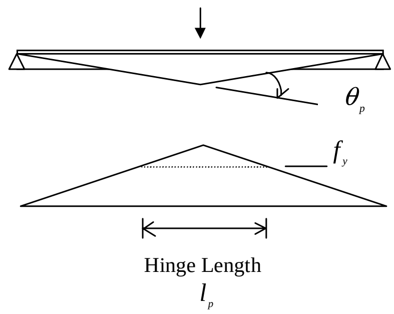

Figure 2-24: Conditions of an Under-Reinforced Beam at Failure

Plastic rotation ductility (empirical) μ_{rot} and rotation angle Θ_p

$$\mu_{rot} = \varepsilon_u^{\,0.75} \left(\frac{f_{st}}{f_y} \right)^{0.9} \qquad (17)$$

$$\Theta_p = \beta_{rot} \frac{(\varepsilon_u - \varepsilon_y) l_p}{d - x} \qquad (18)$$

are a function of $\frac{l_p}{l} = 1 - \frac{f_y}{f_{st}}$ (l_p represents the plastic length of the hinges where the steel stress exceeds the yield), while $\frac{f_{st}}{f_y}$ is the ratio ultimate stress to yield stress. So, it depends on element type and slenderness, on the neutral axis/effective depth ratio $\frac{x}{d}$, on the rebar strain ε_u, and on bond (Beeby, A. K.; 1997 and Bachmann, H.; 1997). If bending is dominant, the ultimate deformation is not correlated to system ductility, but depends on the stress-strain relationship of concrete and on the post-yield characteristics of the rebars. In reinforced concrete many cracks grow and stabilize slowly in a long hinge region once a certain-depth is reached. This is not the case for brittle plaster or light masonry, where few major cracks immediately cross the entire section. They significantly affect their micro-structure leading to excessive elastic deformation (Amara, K.B. 1996 and Frangopol; 1997). The available curvature ductility

$$\mu_c = \frac{l^2}{3l_p(l - 0.5l_p)}(\mu_d - 1) + 1 \qquad (19)$$

depends on l_p, amount of confining rebars (displacement ductility lower)

$$\mu_{d,beam} = 1 + (\mu_c - 1)\frac{2h}{l} \quad and \quad \mu_{d,column} = 1 + (\mu_c - 1)\frac{3h}{nl} \qquad (20)$$

where $\mu_c = (2 \ to \ 4)\mu_d$ (Park, R.; 1996, Bachmann, H.; 1997 and Keintzel; 1998). If the stirrups are effectively anchored and their spacings meet the code requirements, one may assume $\mu_d = 6$, otherwise $\mu_d < 2$. As it is not always possible to predetermine the sequence, in which two plastic hinges fail, all possibilities have to be checked using e.g. a fault-tree (see 4.1). To investigate if plastic hinges occur in beams or in columns -usually being revealed by an abrupt change in story drift-, a Sway potential index

$$S = \frac{\sum(M_{bl} + M_{br})}{\sum(M_{ca} + M_{cb})} \qquad (21)$$

may represent the ratio at a horizontal level of beam flexural strength on the left and right of a joint to column flexural strength above and below the joint (Park, R.; 1996 and Figure 2-25).

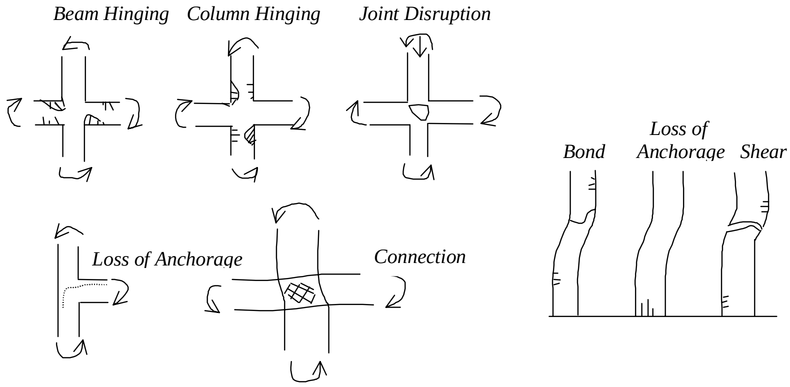

Figure 2-25: Ductility-Related Failure Modes (Folic, R. J.; 1991)

Whereas a soft story occurs due to stiffness discontinuity, a weak story occurs due to strength discontinuity with hinge formation in weak columns, and not in the strong long-span girders. For buildings up to five stories the ductility demand is roughly equal over the height. For taller buildings it is larger in the upper and lower stories and decreases in the middle stories. System flexibility depends on a structure's natural eigenperiod and may adequately be characterized using empirical formulas

$$T = 0.35H^{0.75},\ T = \frac{H}{50} \quad \text{or better} \quad T = \frac{0.06 \text{ to } 0.09H}{\sqrt{B}} \qquad (22)$$

with H and B for building height and breadth respectively, both in meters. For moment resistant frames the formula $T = 0.1N$ with N for the number of stories is sufficient (Bernal, D.; 1992 / Kinney, G. F. et altri; 1988, Chaallal, O. et altri; 1997, Chaallal, O. et altri; 1997 and FEMA-178; 1992). Buildings with H between 60m and 250m are dealt with (Rybicki; 1978)

$$T \cong 2.5\left(\frac{100}{H}\right)^{-1.6} \qquad (23)$$

For a top- and bottom hinged column one may approximately assume an eigenfrequency of 20 Hz, and 100 Hz if the bottom is built-in and the top hinged. Note, that the degree of fixing may-just as an elastic bearing never be described with crisp values. Here, again fuzzy-logic may serve for a quantification of uncertainty.

Every structural element will participate in resisting vertical and lateral forces proportionally to its rigidity (strength) relative to other components, provided that the existing bracing is complete. Weak diaphragms might attract forces they are not capable to sustain. The same applies for welded pre-cast elements as a consequence of unintended frame action. Moment resisting frames predominantly deform in brittle shear leading to large story drifts (case b), stirrup yielding and inclined compression with crushing concrete. Compared to slender columns these fail in shear and not in flexure. Here, in addition to an appropriate anchorage of rebars and steel yield strength column ties and beam stirrups should confine concrete. Shear walls or diagonally braced frames, if detailed in a ductile manner (with warning), deform in a bending mode (case a) with prior yielding of the longitudinal rebars (see Figure 2-26). Very small and very large rebars percentages result in a brittle behaviour, while medium combinations of stiffness and strength lead to ductility.

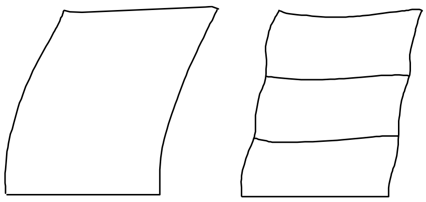

Figure 2-26: Failure Mechanisms dependent on the Structural Configuration

The shear and bond strength of both members and joints have significant influence on the global lateral force capacity and on their energy dissipation capacity. For an adequate resistance towards repeated and irreversible cyclic deformations. Any inelastic behaviour should occur in the members rather than in the connections, especially in case of eccentricities (FEMA 178; 1992). In slender walls high bending forces require substantial boundary reinforcement at wall ends, especially adjacent to openings and at corners. Here, adequate transverse reinforcement at the level of openings must compensate insufficiently anchored longitudinal rebars. Long walls which stop at an upper level instead of continuing to the foundation are vulnerable to shear forces. These are transferred to a strut or a specially detailed diaphragm, while the overturning forces in any case continue down to the supporting columns. “Short columns” with $\frac{d}{l} > 2.5$ refer to infill walls which stop short of beams with openings between their top and the frame beam above (König, G. et altri; 1999 and FEMA 178-1992). The Murrah Federal Building in Oklahoma City did not collapse due to a defect of one single element, but rather due to its inadequate topology. Only one in three of the buildings’ outer columns was supported on its own foundation (Starossek, U.; 1999). The other columns rested on a transfer girder that ran across the face of the structure on the second floor, and the loads having been assigned to a failing column could not be redistributed to neighbouring columns. Thus, failure was not remained locally limited, but spread further throughout the whole structure without a previous warning. If all columns had been extended to the foundation level, overall safety would have been higher and, the consequences of the bomb attack would have been less.

Redundancy represents the availability of ordinarily not required capacity, i.e. the number of critical regions that need to be yielding to produce collapse. It may be active (sharing loads) or stand-by (some components become active only if others fail). A deterministic redundancy measure is the indeterminacy

$$I = F - E \qquad (24)$$

with F and E representing the number of unknown reactive forces and independent equilibrium equations, respectively, or better

$$R_{determ} = \frac{L_{intact}}{L_{intact} - L_{dam}} \qquad (25)$$

with L_{intact} for the ultimate strength of the intact system (Frangopol, D.; 1986). A single evaluation of a cross-section at some selected areas does not inform about instability after hinge formation. Uncertainties in member capacities, loads or individual risk levels are superimposed by those related to system configuration. Steel and concrete strength uncertainties in the key elements may very well be reduced by additional laboratory tests. A larger scatter, however, has to be accepted regarding the displacement and rotation ductility (Bertero, R.D. et altri; 1999). So, decisions about closing, repairing or replacing do depend on the deterministic, but also on the probabilistic redundancy with respect to failure of the weakest member being defined as (Frangopol, D.; 1992)

$$R_{prob} = \frac{\beta_{syst-coll} - \beta_{weak}}{\beta_{syst-coll}} \qquad (26).$$

Global instability and torsion are caused if the centers of gravity and strength (rigidity) are far from each other or, if there are discontinuities in geometry (external or reentrant corners, setbacks). Whereas stiffness asymmetry (unequal distribution of lateral forces to shear walls or differing materials within a structural element) influences the elastic response within story shear or torque only producing translational mechanisms, strength asymmetry always produces an inelastic response with torsional mechanisms. Here the deformation demand concentrates in resisting planes far away from the center and so, is strongly influenced by the shape of the story shear and torque surface. Any building with almost any amount of torsion can be designed to meet code forces, but not to perform well in accidental load situations. Its dissipation capacity may not only be affected by horizontal irregularities (in plan), but also by vertical discontinuities (in elevation). The latter refer to centers of different floors not approximately lying on vertical lines, to elements not being continuous to the foundation or to open storefront. Horizontal and vertical irregularities in strength (30%), stiffness (20%), mass (50%) and geometry refer to dimensions of the force resisting system and not the dimensions of the envelope (Chopra, A.K. et altri; 1996 and Figure 2-27).

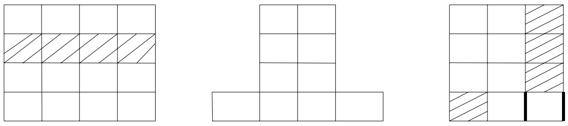

Figure 2-27 Discontinuities in Mass, Geometry, Stiffness (FEMA 178-1992)

In plan asymmetric buildings those elements remote from the center of twist are required to provide a higher displacement ductility than those in regular buildings. The system ductility demand for structures with limited torsional restraint should therefore be limited to (Pauley, T.; 1998):

$$\mu_\Delta = \mu_{\Delta 2\max} - \frac{\lambda_1 - 1}{\sigma(1+\psi)} \qquad (27)$$

$$\textit{with } \mu_{\Delta 2\max} = \frac{u_{\max}}{u_{\text{first yield}}} \qquad (28)$$

$\mu_{\Delta 2\,max}$ represents the maximum ductility demand expected to develop in element 2, while $\sigma = 6\%$ is the post-yield stiffness coefficient when estimating the inelastic torsional response (strength dependent). Whereas ψ represents a geometric system parameter dependent on the arrangement of the lateral forces' resisting elements in a torsionally unrestrained system, $\lambda_1 < \sigma(\mu_{\Delta 2\max} - 1)$ represent an upper limit to excess strength of element *1* beyond which, for a given post-yield stiffness of element *2*, it can never yield. Not only strength and effective stiffness of single members are important, but also those with respect to the whole structure. The global critical load N_{crit} characterizes structural health or performance and the danger of pure sway, pure torsion or combined sway-torsion also referred to overturning/elastic buckling of the whole system. The ratio $\lambda = \frac{N}{N_{cr}}$ mustn't be reduced by more than 10%. The eigenvalue method is replaced by the equivalent column approach (Zalka, , K. A. et altri; 1996):

- moment of inertia of the equivalent column $I_x = \sum I_{x,i}$, $I_y = \sum I_{y,i}$ as sum of individual inertia moment of each bracing element (principal directions)
- bending torsional constant $\mathrm{I_w} = \sum \mathrm{I_{x,i}}^2 x_i^2 + \sum \mathrm{I_{y,i}}^2 y_i^2$ here the stiffness center of the bracing has its origin in the „gravity center“ of the individual moments of inertia of the bracing elements with
- $N_{cr,x} = \dfrac{7.8\beta_{stories} EI_y}{H^2}$, $N_{cr,y} = \dfrac{7.8\beta_{stories} EI_x}{H^2}$ with H for the height of the building
-

Stories	*1*	*2*	*3*	*4*	*5*	*10*	*30*	*>50*
$\beta_{stories}$	*0.315*	*0.528*	*0.654*	*0.716*	*0.759*	*0.863*	*0.950*	*1.0*

Table 2-4: Factor β dependent on the Number of Stories

$N_{cr,\varphi} = \dfrac{7.8\beta \mathrm{E\,I_\omega}}{\mathrm{F^2 H^2}}$ and $\mathrm{F^2} = \left(\mathrm{L^2 + D^2}\right)/12 + \mathrm{R^2}$ for bending behaviour and

$N_{cr,x} = \dfrac{\pi^2 E I_y}{h^2}$, $N_{cr,y} = \dfrac{\pi^2 E I_x}{h^2}$, $N_{cr,\varphi} = \dfrac{\pi^2 E I_\omega}{F^2 h^2}$ with h for dominant shear.

- $\lambda_{global} = \lambda_x + \lambda_y + \lambda_\varphi$ with $\lambda_i = \dfrac{N}{N_{crit,i}}$;
- for x-symmetry $\lambda_{global} = max\,of\ \lambda_y\ and\ \left(\lambda_y + \lambda_\varphi\right)$ is sufficient

L, B, R represent length, breadth, distance stiffness-gravity center, respectively.

Ductility, dissipation and strength are not only required in the elements of a buildings’ superstructure but even more in those of its substructure, in the connections of the superstructure to the substructure and in all components of the associated load paths. Otherwise, e.g. a shear wall instead of yielding as intended may rotate like a rigid body. Often the performance of foundation becomes obvious by observing supported elements in the local area. In contrast to spread footings, piles and piers need special hoops or ties immediately beneath the caps and confining transverse reinforcement to provide sufficient flexural strength and ductility at the connections. High bending moments change soil stiffness or lead to settlement thus significantly compromising integrity.

Relieving e.g. pressures from explosion, nonstructural elements may attenuate or contribute to the recovery of the building to its intended function certainly being to the benefit of the occupants (Carper, K. L.; 1998). Cracks in the panel materials indicate structural distress and, exterior walls revealing corrosion, erosion or temperature movement might trigger a system to collapse in a new incident. The buffer effect on lateral load resistance of brittle sandwich panels may very well be simulated by a hysteretic model. Being linear elastic until peak-loading is reached, the introduction of further deformation leads to steady crushing under nearly constant load and, once being fully compacted, to a rapid load increase (identification point). Finally unloading (after steady crushing has started) produces minimal recovery. Though many nonstructural components are directly connected to the structural system, their behaviour is largely ignored in conventional analyses. Often, ignoring them is considered to be conservative in certain cases, a quantification of their effects on the structural response may be important. An increase in stiffness would cause a building to attract higher lateral forces. Besides, reductions in the cost of rehabilitation or retrofit may be possible, if the stiffness and strength contributed by nonstructural components could be assessed and was found to be significant. Just as important as the above mentioned topics are equipment and machinery in buildings. A generator e.g. might be able to survive if lightweight shields or assemblies minimize debris and fire from reaching the generator and its controls.

2.5 Risk Analysis and Cost Minimization

The risk of structural failure depends on damage extent and its possibility of occurrence, but also on the significance or value of relevant components. Risk analysis may therefore be defined as detection of weak areas or structural faults and investigation of system topology to identify consequences of damage induced component loss. A well known approach is not to calculate risk directly, but to determine failure probability of a structure with respect to point failure, section or system failure. However, insurance companies require additional information which cannot be obtained by an isolated application of the above mentioned method (see chapter 4.3). Even, if money was of marginal significance, the component importance and its value would be indispensable aspects.

With more and more reduced financial resources the latter become more and more important with respect to cost minimization. So, in addition to reliability estimation, engineers are required to minimize costs with the constraint of a predefined target reliability. Numerous influences may lead to a reduction of structural reliability itself representing an optimization problem to find a robust solution even for uncertain parameters. In the same way as codes are required for draft and design, for existing structures guidelines on how to decide about their further use are needed. Since type and location of damage may -dependent on the loading characteristics and the loading scenario- lead to a different structural behaviour, deterministic analyses are not sufficient. The higher the safety demand of society, the better risk analyses should be.

Risk analysis involves the prediction of an adverse event and its effect on structural capacity finding out when the limit state is reached (Hwang, H. et altri.; 1987 and Yao, J.T.P.; 1996). It always begins with a qualitative judgement performed by the decision makers. In two processes these should integrate and weight not only safety, but also health and welfare of citizenry. After having derived the best option of action from a set of alternative options, they try to compute uncertainty with the help of probabilistic or logical tools. Here, the engineers must present the relevant factors in the clearest possible terms and in an easily understood form for the audience addressed. The public needs to have a picture painted in order to finally comprehend the significance of essential elements and, thereby arrive at an appropriate option choice.

For example, a mobile bridge being used only over a short time one may accept a higher risk than for a permanently used bridge. Traditionally strength has been estimated by practical experience or semi-stochastic analysis considering not only the scatter of data or socio-cultural attitudes, but also their time-dependent development. In contrast to air-crafts, civil structures have to fulfil serviceability over some 50 years. Their failure can have catastrophic consequences and over-conservatism lead to extremely high costs. So, not only for nuclear power plants or offshore structures, a detailed safety assessment is required. However, in no engineering product 100% safety is obtainable and, a certain amount of implicit risk with respect to economic values and human life has to be accepted. The risk of a civil structure may be small, but it is greater than zero.

Already in 1924 Freudenthal proposed to incorporate decision theory in safety and reliability calculations (Freudenthal, A.M.; 1956). Reliability targets or safety indices depend on the relative cost of safety measures, potential repair and operation or user costs and, on the possible time out of service leading to a lack of occupancy, a loss of wages, a potential loss of manufacturing capability and other multiplier effects (Amman, W, 2000, Kiefer, D; 2000, Wörner, J.D; 2000, Khanna, P. et altri; 1992 and Preyssl, C. et altri; 1992). Public discussions reveal a target risk $P_{target,f}$ / reliability index β (Brühwiler, E; 2000).

$$10^{-6}\,(safety) < P_{target,f} < 3\times 10^{-4}\,(serviceability) \qquad (29)$$

$$4.7\,(safety) > \beta > 3.5\,(serviceability) \quad with \quad \beta = -log_{10} P_f \;\; approximately \qquad (30)$$

In addition to the technical and economic aspects, juridical, political, ecological and, last but not least emotional issues have to be considered. Several approaches attempted to relate damage extent and value (see Table 2-5).

Value (→ increasing)			
↑ ***Damage Extent***	*Medium*	*High*	*High*
	Low	*Medium*	*High*
	Low	*Low*	*Medium*

Table 2-5:Relationship - Damage Extent / Value (Brühwiler, E; 2000)

Although local failure, e.g. occurring due to the yielding of rebars in a bridge deck or due to a plastification in a redundant system, does not threaten life, it should be limited as far as possible, especially if one wants to keep the costs for safety measures low. In case of a global failure due to e.g. the breaking of an important bearing, life in general is threatened. A regional failure due to the loss of a structural component does not necessarily endanger life (see Table 2-6).

Target Indices for Safety		**Risk category (Value)**		
		High	*Medium*	*Low*
Type of failure	*global*	*4.7*	*4.7*	*4.1*
	regional	*4.7*	*4.1*	*3.5*
	Local	*4.1*	*3.5*	*3.5*

Table 2-6: Reliability Indices for Safety with Respect to the Failure Type (Brühwiler, E.; 2000):

On the one hand risks have to be restricted to an acceptable limit with cost constraint (reliability optimization). Here, for every possibility minimum costs are associated with procurement time operation, maintenance, inspection, overhaul, repair downtime, disposal events and destruction comparing the costs for other combinations. On the other hand time-variant or time-invariant repair costs have to be limited together with weight, material, cross-sections, member sizes, defects and deadlines (reliability constraint). An objective function may be

$$C_{total} = C_{const} + C(\beta) + \sum P_{costs,i} C_{repair,i} \rightarrow min \qquad (31)$$

with $C(\beta)$ for the increasing costs dependent on the safety factor β, P_{costsi} for the probability of necessary repair, and $C_{repairi}$ for the repair costs (see Figure 2-28).

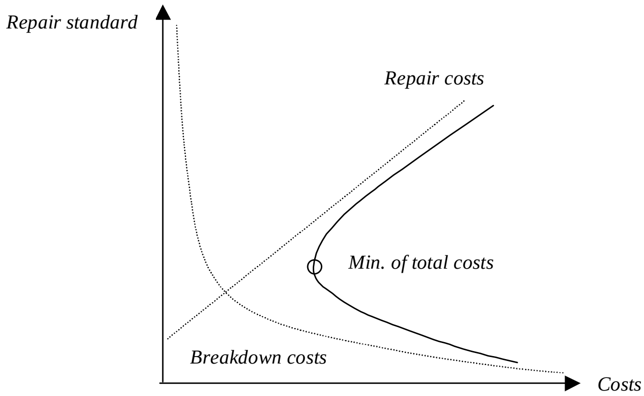

Figure 2-28: Expected Total Costs Dependent on Structural Parameters (Frecker; 1999)

When deciding about retrofit additionally to immediate benefits, evaluate likely benefits and costs throughout the residual life (see Figure 2-29).

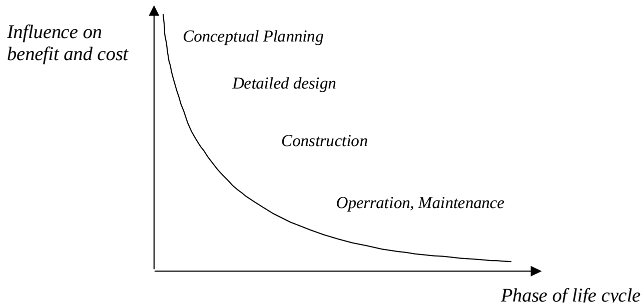

Figure 2-29: Influence on benefit and cost - life cycle (Mori, Y et altri.; 1994)

Consider that break down or not repairing structure leads to environmental problems and forces an occupier to resort to other owners in turn requiring a payment, if they cannot continue to carry on business. A quantification of the relative costs is given by the "law of five" (see Table 2-7). The reference type and time of usage for structures are summarized in Table 2-8).

Actual phases within a civilian structural life	**Relative costs leading to a specified quality**
Conceptional planning, design (expert judgement), construction (logical models)	*1*
Production phase and use of the structure (qualitative PSA and empirical data)	*5*
Small repair (maintenance)	*25*
Extensive repair (changes of bearing structure), Possibly destruction (empirical data)	*125*

Table 2-7: Relative Costs According -"Law of Five" (Mele, M. et altri; 1999)

Intended time of usage	***Range of application***	***Additional remarks***
<5	*Temporary structures*	*Temporary theater-of-operations structures*
<25	*Replaceable structural elements*	*Supports, joints subject to extensive corrosion/fatigue/degradation*
50	*Usual structures*	
>100	*Monuments, historical buildings of high value, bridges and other engineering structures*	

Table 2-8: Reference Type and Time of Usage (Rackwitz, R.; 1997)

Every budget or service evaluation considers existing resources (information, labour, material, plant and essential services), but also which stakeholder group is most in need (owner or occupier). The owner buys resources to repair infrastructure (efficiency) in order to supply service at minimum costs (economy) for occupiers fit for the intended purpose (efficacy) and add value (equity). Decision variable pairs are illustrated in Figure 2-30):

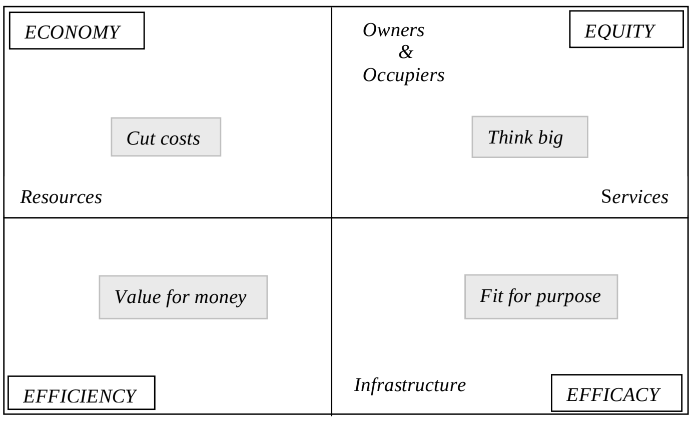

Figure 2-30: Objectives of Owners and Occupiers and Possible Interactions (Frecker, G.B.; 1999)

With a proof load level defined according to specific requirements of safety (80-85% of the serviceability), serviceability and durability, a limit state function has to be established using both measured and analytical data. A lack of safety comes from overall instability (overturning), sliding (strain localization), plastic deformation (yielding), differential settlement, brittle compression or connection failure (distortion due to separation), ductile flexure (tensile fracture) and fatigue. Serviceability is related to everyday needs or to the intended function considering economy, reparability after vibration, excessive elastic deformation, rotation of a component relative to others or to its original shape. Deflection can be categorized in sensory acceptability (vibrating floors can distress occupants), serviceability effect (vibration sensitive laboratory equipment on structural elements, brittle partition walls which must meet a floor that is not in level), and safety requirements (direct effect on structural elements causing instability or high stress). Preparative studies for the European codes additionally discerning failure probability with respect to one year (in brackets) and 50 years (in bolt print) are complemented by Dutch requirements for a reference period of one year (König, G.; 2000, Vrouwenvelder, T.; 2000 and Bergmeister, K.; 2000).

For structures of middle age NDE seeks evidence of deterioration due to corrosion of the embedded reinforcement after chloride penetration or due to alkali-silica reaction. Both refer to long-term and gradual changes, while defects are found in the conception or in the construction/manufacture technology such as poor detailing, improper placement of reinforcement, deficient steel quality or substandard concrete. Time-dependent reliability is substantially affected by load or environmental processes. Environmental stressors (sulfate attack, expansive aggregate reactions) may trigger a damaged member into failure or at least compromise its integrity, if accumulated or synergetic effects have been disregarded previously or not considered to require remedial action. Such calendar usage leads to continuous material degradation, while technical usage from loading leads to overturning, shear cracks or fatigue. Table 2-9 shows failure mode analysis including defects, deterioration, real damage and performance.

Defect	***Deterioration***	***Damage***	***Performance***
Excessive flexibility *Deficient interface detail* *Lack of concrete cover*	*Debonding,* *vertical/lateral coupling* *Carbonatization* *Corrosion* *Slow Crack Growth*	*Cracking/delamination* *Scaling, spalling* *Fatigue* *Loss of Section* *Bearing drift* *unseating*	*Reduced serviceability* *Reduced service life* *Increased lifecycle cost* *Increased vulnerability*

Table 2-9: Failure-Mode Analysis (Aktan, A.E. et altri; 1996)

When calculating risk P_f, this can be reduced in dependence of the detection probability P_{det}, if experienced engineers are available (Brühwiler, E.; 2000):

$$P_{f,red} = P_f - P_f P_{det} \qquad (32)$$

If heavy damage can be detected at the right time, the represented danger is less. Dependent on the damage extent $\eta(t)$ and on the inspection technique/scheduling/quality σ_{insp}, the detection probability may be calculated with

$$P_{det} = \phi\left(\frac{\eta(t)-\eta_{0.5}}{\sigma_{insp}}\right) \quad (33).$$

ϕ represents the Probability Density Function of the damage extent η at time t, while the minimum damage extent at which a detection is possible. The damage extent η dependent on time t may be calculated with

$$\eta(t) = \frac{A_{initial} - A(t)}{A_{initial}} \quad with \quad A(t) = A_{initial} - 0.102t \quad (34)$$

It represents an average value for deterioration with time (Frangopol, D.; 1999, Frangopol, D.; 1997). $A(t)$ is the remaining cross-section area after time t during which the structural element was subjected to destruction (approximately 0.102 t). The probability that damage is detected may be expressed with

$$P_{det} = 1 - \left\{1 + \frac{P_{\eta ultim}}{1-P_{\eta ultim}}\left(\frac{2\eta}{\eta_{ultim}} - 1\right)^{p}\right\}^{-1} \quad for \quad \eta > 0.5\eta_{ultim} \quad (35)$$

$$p_{det} = 0 \quad for \quad \eta < 0.5\eta_{ultim} \quad (36)$$

with η_{ultim} for the limit state for damage, and $p_{ultim\,\eta}$ for the damage detection probability for $\eta = \eta_{ultim}$ (Könke, C; 1999a and Figure 2-31).

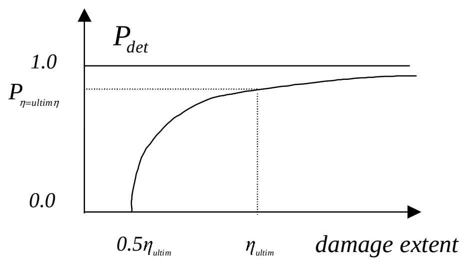

Figure 2-31: Damage Detection Probability dependent on the Damage Extent

More exact is to differentiate between concrete and steel using the formulas

$$A_s(t) = N\frac{\pi(D_0 - k_s t)^2}{4} \quad and \quad A_B(t) = h_0 - k_c t \qquad (37)$$

(Könke, C.; 1999b, Petryna, Y. et altri; 1999 and Krajcinovic, D.; 1996). Here N, D_0, k_s and t denote the number, initial diameter, rebar corrosion rate in cm/year and corrosion time in years, respectively. k_c and h_0 indicate concrete corrosion rate in cm/year and initial dimension, respectively (see Figure 2-32).

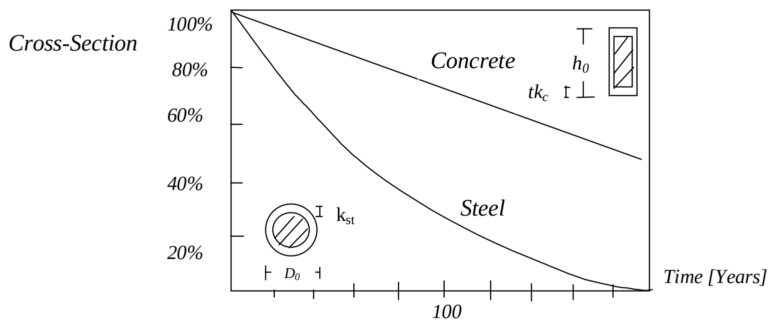

Figure 2-32: Corrosion Rate of Steel and Concrete

The effort required to refine analytical techniques wouldn't guarantee a reduction of partial safety factors, since min or max guaranteed parameter values do not necessarily produce the worse structural behaviour and even may adversely affect integrity. Although dynamic actions vary with time and location thus complicating a description of the deformation - stress relation, they are used to determine the global health of structures (Mukherjee, A. 1997). A realistic lifetime orientated optimization requires the identification of structural condition stability[10] considering time-dependent nonlinear material properties randomly distributed in space together with random loading processes, and different probable load combinations fluctuating in time and space.

Deterministic analyses, primarily developed for structures with a high damage potential, intend to determine past load histories for each member having undergone inelastic deformation, and for the building as a whole. This may be done by quality control of an element with unknown resistance, strain measurement, crack detection, or by load tests on an element whose reliability has already been computed (resistance is higher than loading).

Here every cause has to be associated, with certainty, to an effect and viceverse, without any incomprehensible residues left over. Determinism is an idealization, for it accepts that the phenomena will evolve in one way only requiring the engineer to be sure about the type and location of the failure modes. So, many load paths have to be investigated because one never knows

[10] probability that a predefined limit state (from time-history and fatigue estimated values, above or below the relevant variable are not situated) is reached at least once during expected service of life.

which is dominant (R. Gori and E. Muneratti, 1998) Damage does not only depend on strength at some critical section or on static indetermance, but rather is a function of statistical correlation between the varying resistance load and resistance parameters (Reinhorn, A; 1997, O'Connor, J. M. et altri; 1995 and Ellingwood, B.; 1987). Loading and structural response may be formulated in a deterministic and probabilistic sense (see Figure 2-33):

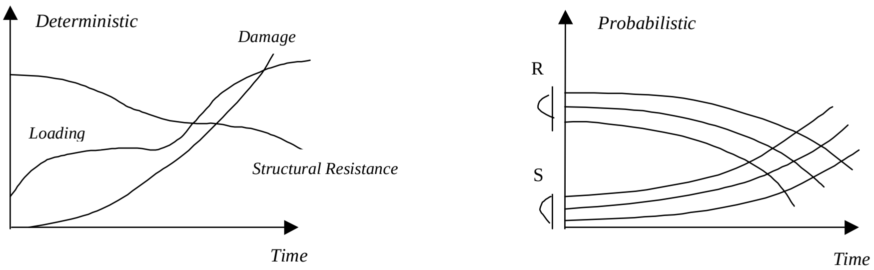

Figure 2-33: Evolution of Loading and Structural Response in a Deterministic and Probabilistic Sense (C. Könke; 1999a)

Since the relevant failure mode cannot be predicted explicitly and, different loading conditions lead to a different failure sequence, additional stochastic analyses are required. Instead of explicit here, implicit Limit State Functions illustrate the deviation between actual and desired situation with respect to all possible load paths or scenarios and informing about the failure probability

$$P_{f,stat} = f_T\left[t, g(x)\right] \quad with \quad g(x) = g_R - g_S \qquad (38).$$

Whereas g_R represents a function of empirical limit data or values point-wise obtained from FE analysis (Structural Capacity / Resistance), g_S indicates the actual force distribution (Stress). Here, the damage extent including failure modes such as flexure, shear, gradual material degradation, debonding or tension stiffening are considered (damage consequence and element importance are neglected). Since there are never enough experimental data, other information sources such as literature or expert opinion have to be consulted.

Due to the fact that polynomials are multidimensional and the variables correlated, a closed-form LSF allowing an exact integration (level 1 method) of $g(x) < 0$ cannot be defined. So, simulations with a variation in type and duration of loading not only capturing the necessary characteristics in a deterministic sense, but also matching specified mean and standard deviation „targets" must help achieve a low annual probability of occurrence. The most famous method to solve the LSF, Monte Carlo Simulations whose generated data are inserted calculating n-times the results. In addition to the relatively exact, but time-consuming MCS (Latin Hypercube sampling can reduce the number of necessary simulations) there are also Point Estimation Methods such as the Tailor Series or the Finite Difference Analyis (kind of sensitivity analysis) and the approximate methods: whereas First Order Reliability and Second Order Reliability analyses (level 2) describe linear LS, Adaptive MCS or Importance Sampling[11] (level 3) describe nonlinear LS's (Daponte, P. et altri; 1998, Schueller, G.I.; 1997 and Möller, B. et altri; 1999). Probability Distribution Functions relating input motions from past investigations to the structural response have to be carefully selected. Due to the rapidly increasing computing time for probabilistic simulations with each additional random parameter, they should be reduced as far as possible. The application of the Response Surface Method is faster, since it only determines the failure condition point-wise (Schueller, G.I., 1997 and Kraetzig & Petryna; 1999).

The computational effort for random data generation in MCS often is disproportionate to the improvement of the results. Besides, non-statistical (indescribable) uncertainties cannot be quantified. Coming from the temporal and spatial correlation of all the function parameters with respect to material, geometry and loads, these might lead to an under-determination of P_f. However, there are not sufficient data of load effects and of response processes. The number of necessary simulations strongly depends on the expected risk. For $P_f = 10^{-3} - 10^{-6}$ at least 10^6 simulations would be required in order to keep the coefficient of variation $\sigma(x)$ below 10% (Schueller, G.J; 1997a and Pfanner, D. et altri; 1999). As the costs increase with the number of realizations r and the square number of the failure modes, usually less than 100 realizations (uncertainty decreases in proportion to $r^{-0.5}$) are performed looking at the sum of virtual work of already yielded hinges (those cross sections with maximum P_f), and at the resulting stiffness change (see Figure 2-34).

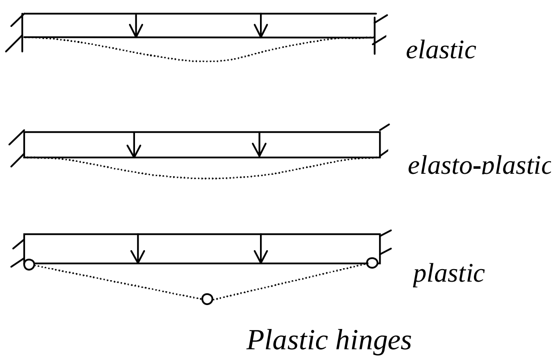

Figure 2-34: Elastic and Plastic Structural Response

[11] reduces the number of simulations to some degree weighting the relevant design value by density

$$m(x) = 0.5(x_{high} + x_{low}) \quad \text{or better} \quad \sigma(x) = \sqrt{\frac{\sum_{i=1}^{n}(x_i - m(x))^2}{n-1}} \quad \text{and} \quad m(x) = \frac{\sum_{i=1}^{n} x_i}{n} \qquad (39)$$
$$\sigma(x) = 0.25(x_{high} - x_{low})$$

$m(x)$ represents a failure modes' mean value primarily being decisive for system failure, and $\sigma(x)$ its variance (Achenbach, J.D.; 1998, Crandall, S. H., Zon, W.Q.; 1983, Schueller, G. J.; 1997 and Bronstein/Semendajew; 1976). For its approximate determination >30 simulations are sufficient (Schueller, G.J; 2000). The standard normal distribution is characterized by the safety factor

$$\beta = -\phi^{-1}(P_{f,stat}) \quad \text{or} \quad P_{f,stat} = \phi(-\beta) = 1 - \phi(\beta) \quad \text{for} \quad m(x) = 0 \quad \text{and} \quad \sigma(x) = 1 \qquad (40).$$

Here, $\beta = \frac{m(x)}{\sigma(x)}$ with $\beta \cong -log_{10} P_f$ represents the number of standard deviations σ_m of the basic variable x which fit between the point of origin and the mean (see Figure 2-35). Since the structural resistance g_R may be below the imposed loading g_S, the LSF $g(x)$ allows calculating the failure probability $P_{f.stat}$ by integrating the joint probability density function[12] $\phi = f_x(x)$ using numerical procedures such as MCS. Since $lim\,\phi[g(x)] = 1$, one may deduce $\int_{-\infty}^{+\infty} \phi[g(x)]dx = 1$. For continuous distributions the probability at a single point is zero. The probability that the LSF is negative (integral between the relevant two points) means that the existing stress exceeds the allowable stress.

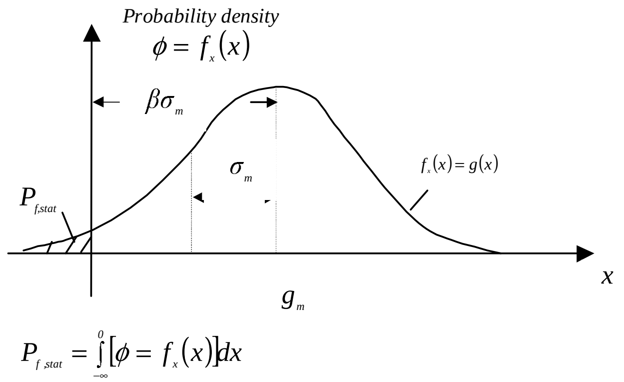

Figure 2-35: Probability Density ϕ versus LSF $g(x)$ for the Difference between Existing and Allowable Stress

[12] defines probability distributions, i.e. represents the probability that a variate has the value $g(x)$

Structural performance may be described by various limit states. There is not only stress, but among others there are the failure modes excessive deflection, bond failure, buckling, interstory drift (local -horizontal displacement of one story), system drift (global -horizontal displacement of the whole system).

	Parameter	*Scatter*
Concrete	*E*	*0.1*
	β_D	*0.14 (0.19 for* τ *)*
Steal	*E*	*0.04*
	Yielding	*0.05*
Geometry	*Height-Length*	*5 mm*
Loading	*Constant*	*0.05-0.15*
	Variable	*0.5-0.8*
Model	*Load-Effect*	*0.1*
	Resistance	*0.07*

Table 2-10: Approximate Coefficients of Variation for Several Parameters (Rosowsky, D.; 1997, van Grabe, W. et altri; 1997, Eibl, J; 2000)

Table 2-10 summarizes approximate coefficients of variation for several independent random variables x_i. Polynomials represent a mathematical tool to characterize complex LSF. With the standard deviation $\sigma(x)$ only exerting a small influence, first order polynomials of the form

$$g(x) = a + \sum_{i=1}^{n} b_i x_i \qquad (41)$$

with $b_1 = 1$ may adequately describe at least the mean value $m(x)$ (Schueller, G. J.; 1997, Enright, M. P. et altr; 1998; Grabe, W. et altri; 1997 and Möller, B. et altri; 2000). Complex LSF are linearized using polynomials of the second degree

$$g(x) = a + \sum_{i=1}^{n} b_i x_i + \sum_{i=1}^{n} c_i x_i^2 \qquad (42)$$

with $b_1 = 1$ and $c_1 = 0$ to avoid the trivial solution, provided that $P_f < 10^{-4}$ and the dimensions >15. In order to define the unknown polynomial coefficients a_i, b_i and c_i, perform N calculations with different points (at the limit state).

$$N = n + \frac{n(n+1)}{2} \qquad (43)$$

An increase of N results in a refinement of random fluctuations, while higher order polynomials capture nonlinear process behaviour (Schueller, G.J.; 1997a and Schueller, G.J.; 1999). If there are more points than required to calculate a_i, b_i and c_i, one may apply the method of least-squares minimizing errors. The effectiveness of reliability analyses may additionally be increased by the execution of preliminary sensitivity analyses. They indicate parameters that exert the most influence and facilitate an examination of sources of uncertainty together with their impact on risk. In order to improve the polynomials themselves one may apply the least-squares method. The whole space of a LSF is subdivided in a safe and collapse area

$$g(x) = min\lfloor g_1(x), g_2(x) g_3(x) .... g_{n_{modi}}(x) \rfloor = \xi g_R - g_S(x) \leq 0 \qquad (44)$$ [13]

Here ξ represents probabilistic (random) uncertainty, X the state vector and g_i all relevant failure modes each requiring a complete structural analysis (Schueller, G. J.; 1999, Zilch, K. et altri; 1999 and Könke, C.; 1999a). With the performance functions of the various failure modes defined, the corresponding risk of single structural elements can be calculated. However, since the above mentioned tools neither take into account the importance of the relevant structural components nor the expected frequency of undesirable events, another method will be proposed in chapter 4.3.

2.6 Consideration of Uncertainty

In addition to material, loading or soil related uncertainties there are also those related to the selected boundary conditions, the applied models and related to man himself.

Whereas parametric techniques adjust parameters to the measured response assuming a constant model topology with predefined constraints ("White-Box" model), non-parametric techniques independent from the true representation of a structural system directly relate excitation and response, and so are not corrupted by modeling errors such as neuronal networks, however representing a black-box. The results of both techniques look quite promising, but there are still some problems that need to be resolved. The author recommends the application of fuzzy-logic in either cases to overcome the limitations related to the fact of statistically insufficient data (parameter is not obligatorily within a specified range) and of vagueness in interpretation (parameter certainly is within a specified range).

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[13] For deteriorating structures it depends also on the random time T, i.e. $g(x,T) \leq 0$

In contrast to the Bayesian[14] approach which, in condition assessment only rarely converges in reliable results, the recursive least-squares require minimal expertise in order to converge in medium reliable results (Ghanem, R. et altri; 1995). Continuously subdividing[15] suspicious group parameters until all coexisting members are extracted they indicate which parameters of which system elements produce a large effect in the expected direction of modification. Though insensitive members may be affected as well (larger variance), for a preliminary estimate, it is usual to apply sensitivity factors dependent on the partial derivative

$$sensitivity = \frac{\partial \beta}{\partial \xi_i} \quad with \;\; \beta = \sqrt{\sum (x_r)} \qquad (45).$$

Here, x_r represents a first deterministic approximation of the mean value dependent on the variable ξ (Val, D. et altri; 1997 and Yamazaki, F. et altri; 1995). In cognitive studies together all parameter values (e.g. moments of inertia) for each element are changed, before in a check-up study only one value at a time is changed holding constant the others and reversing each change before proceeding to the next parameter (cross-sensitivities which accumulate effects are neglected, Roca, P. et altri; 1997). Also, models with more degrees of freedom do not necessarily lead to a smaller error, but might unreliably influence single observations within parameter estimation. Therefore the subdivision of a model should be discouraged beyond a certain level of redundancy (a high redundancy value is appropriate to find out large errors). The procedure in some way corresponds to the scheme of parametric learning in neuronal networks dependent on the learning rate α and on the performance index Q (Pedrycz, W.; 1993.

$$new\ par\ =\ old\ par - \alpha \frac{\partial Q}{\partial (par)} \qquad (46)$$

After having gathered enough information to construct the model / network topology, the relevant details of the learning scheme can be fully specified. In this way the initial configuration can be optimized by minimizing Q (see also chapter 3.3). Concise information helps understand and describe wholeness thus easing measurement interpretation. In parameter sensitivity studies the system response has to be expanded and the reference model reduced to identify response governing mechanisms with irreducible, uncorrelated variables. In this way it is possible to examine uncertainty, its sources and its impact on reliability. The parameters with the greatest sensitivity to errors require the greatest validation precision and effort

[14] updating theorem
[15] in Artificial Intelligence „cluster analysis“ refers to recursively partitioning a structure into substructures (Helmstad, K. D. et altri; 1997).

to prove consistency (all conclusions can be true at the same time). A modification of structural features of single components and, through dependencies of those of the whole system provides additional information. Although changes of higher magnitude (e.g. 2% stiffness decrease or 10% damping increase) facilitate trend estimation, response parameters should be sufficiently sensitive also to slight perturbations in order to identify the lowest amount of detectable damage. Unfortunately this requirement competes with the need of numerically robust indices insensitive to variance and bias errors.

Since the terms (correct and incorrect) are insufficient definitions and are a part of reality, a weighted fuzzy version of the least-squares

$$\sum_{n=1}^{N}\left(T_i - \sum_{m=1}^{M} x_{nm} w_m\right)^2 \rightarrow min \qquad (47)$$

$$T_i = \frac{\sum_{m=1}^{M} w_m x_m}{\sum_{n=1}^{N} w_n} \qquad (48)$$

with n for the pattern- and m for the output number are used to minimize errors.

In probabilistic models requiring a minimum number of tests with uniform boundary conditions, the frequency of occurrence in general (occurrence of an explicitly defined event) is relevant. Possibilistic models (extent of an event is decisive), however, rely on a new form of information theory which is related to, but independent of both, fuzzy sets and probability theory. Technically all fuzzy sets are possibility distribution functions, but not vice-verse. They specify linguistic scales of numeric information obtained from measurements. Although both models are very similar, they analyze the data in a different way. Probability theory uses artificial intelligence, case-based reasoning, regression analysis, Markov Chain or filtering to converge analytical and experimental data (displacements, accelerations, rotations, strains) upon reliable parameter estimates (stiffness, mass, damping) until a best-match is reached[16], while possibility theory allows a continuous updating of expert knowledge.

Data quality and accuracy may be improved through using more reference instruments in repeated measurements and in combination with a flexible data acquisition software. A great scatter in the data reveals material in-homogenity requiring a more dense grid of sensors. Provided that the errors are statistically independent and not subject to model induced correlation, 12 to 16 sensors are sufficient (Cantieni, R.; 1996). They should be placed where the maximum mode shape amplitude is expected. In this way an averaging is possible by minimizing the influence of runaways, missing or imperfect measurements. The

[16] Modal Assurance Criterion comparing e.g. real and estimated mode shapes through frequency discrepancy (the cosine of angle 1.0 for eigenvectors- is ideal

usual benchmark of feature quality is the empirical error with the always arising question “do the computed results make sense? Dependent on temporal and spatial loading properties, a previous FE analysis is recommended to find out the optimal number and location of the sensors. Many transducers in various positions inform about time (not location) of e.g. an impulse. As already mentioned in the beginning of this chapter, increasing the time-resolution, record length or observation time in a single transducer picking up all mode shapes simultaneously in one position does not improve the frequency determination, but only interpolates within the available frequency information. The product of time and frequency relation is always constant (property of temporal and spatial domain equivalents is complementary. So, only a trncation of time would improve the illustration of individual mode shapes (Dossing, O.; 1998)

Aleatory and epistemic, they play an important role when characterizing classes of data (crisp data and uncertain classes, uncertain data and crisp classes, uncertain data and uncertain classes). Note, that the effects of uncertainty are perpetually present and to avoid fooling ourselves there is no other intelligent way than considering them explicitly. Although many data are crisp in the sense that they are quantifiable, their knowledge base contains qualitativeness. Nevertheless they can be used in the difficult task of interpreting modes of structural behaviour providing information useful to answer to the initial question. Also, stupid conclusions due to the lack of expression capability may be avoided. In classical reliability theory for a limit state >0 the structure is assumed to be safe and in failure state for <0. So, e.g. 10^{-5} would mean reliable and -10^{-5} would mean collapse, although there is no essential difference between them.

To clearly illustrate the difference between random and nonrandom uncertainty the following two laboratory experiments (Blockley, D.I.; 1979): The first experiment is that of measuring the elastic deflection of a simply supported beam at room temperature under a known central point load. The second experiment is to measure the number of cycles to failure of a steel specimen under a known elastic sine wave cyclic fatigue loading. Their result of the first experiment may be accurately predicted using simple beam theory, but the result of the second can be predicted only very approximately. Provided that an appropriate model has been selected, in the first experiment system uncertainty is small and in the second is large. If these two experiments are repeated but this time the applied loads are chosen randomly from known probability density functions, then the uncertainty in any prediction about the outcome of the experiment will be increased and will be a combination of nonrandom and random uncertainty. If the experiments are then transferred to some real structure, then the problem becomes even more difficult because elements of a real structure never correspond to idealized laboratory specimens. Besides, the random variability of the parameters may be difficult to specify because of the difficulty of getting enough data.

Figure 2.36 illustrates uncertainties from the conceptual point of view:

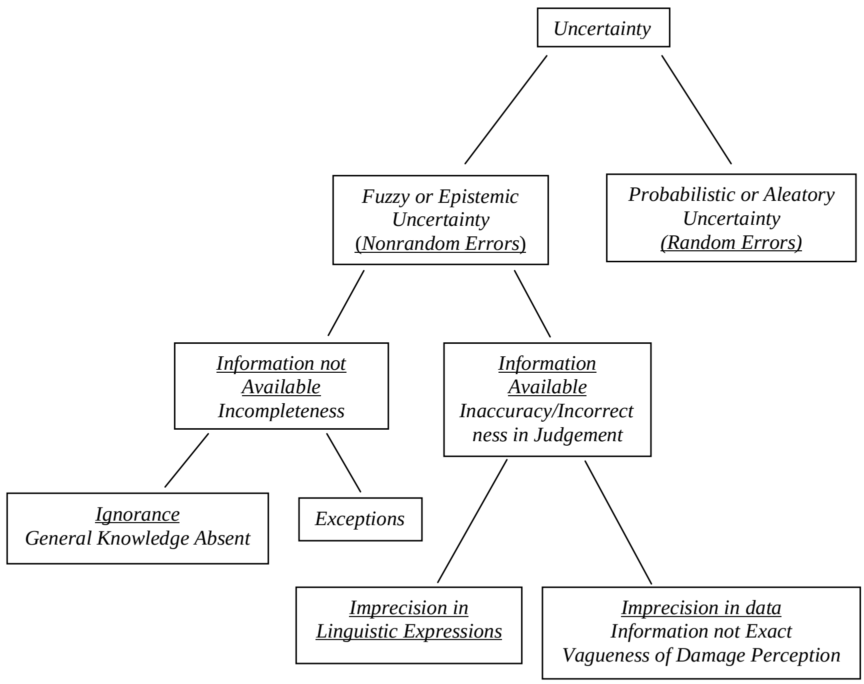


Figure 2-36: Uncertainty from the Conceptual Point of View (Ayyub, B.M. et altri; 1984, Wood, K. L. et altri; 1991, Ellignwood, B.; 1994, Yazici, A; 1999, Kunreuther, H.; 2000 and A. Albert; 2000)

A very good classification of uncertainty with respect to its types and characteristics, respectively, has been presented in a recent publication (Beer, M.; 2002):

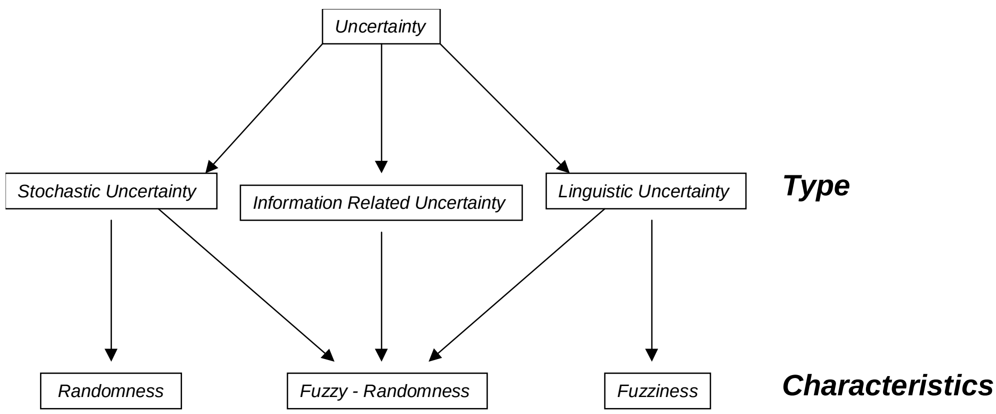


Figure 2-37: Uncertainty Classification with respect to Type / Characteristics (Beer, M.; 2002)

For the mathematical description and quantification of uncertainty there have been developed different tools. Among them probability calculation, interval calculation, convex modeling are used in the same way as assumptions based on subjective probability theory, chaos theory, fuzzy-set-theory and theory of fuzzy-random variables (Beer, M.; 2002). Dependent on the characteristics of uncertainty in turn related to its type and information content (data uncertainty, model uncertainty) different mathematical algorithms are used. Whereas data uncertainty refers to the input variables explicitly being introduced in a model without exerting an influence on it, model uncertainty refers to the abstraction process of reality description (exact input data lead to uncertain results, since the applied algorithms are uncertain). Fuzziness is used to describe both, model and data uncertainty. Fuzzy-randomness, however, predominantly is applied for the description of data uncertainty.

The inherent variability of <u>random errors</u> may be assigned to imprecise data of the test object, namely system geometry, member size (cross-sectional dimensions, slenderness), rebar location, concrete cover, truss action, boundary conditions, temperature, mass density and material (hysteresis, tension stiffening, yield limit) being largely or totally not under the control of an engineer. Microstructural changes not being related to mechanical properties of interest though influencing test results come from roughness, flaw orientation and shape, wind, humidity/moisture content or changing groundwater levels. Random errors also may derive from soil properties (damping), from vague transducer properties (geometry, weight, radiation pattern and impulse response), from disturbances due to imperfect measurement devices such as conversion distortions, undesired reflection, wave absorption, amplification or cable shield in transmission over large distances, electric noise or interference[17] due to induction from power cables near magnetic and electric fields, from vibration of not fixed cables and from thermic noise. Random errors also derive from the measuring procedure itself (sampling in experiments may lead to an incorrect registration of loading, transmission width, arrangement and velocity, ratio of dead load to live load and support movements). Unfortunately one may not always discern noise from relevant signals. Although representing $\frac{1}{3}$ to $\frac{1}{2}$ from the variance errors, bias errors usually tend to be ignored. Both are larger for dynamic and impulsive loads than for static loads.

<u>Nonrandom[18]</u>, i.e. analytical/numerical or modeling <u>errors</u> are systematic and come from nonlinearity, truncation, aliasing (apply anti-aliasing filters!), leakage, clipping, calibration, parametric insensitivity towards important characteristics, or from the conversion of field measurements into parameters for structural analysis (differences in reading translations range up to the factor three). The improper use of complex software leads to false judgements of the failure

[17] interaction of two waves, whose size (amplitude) and degree to which they are in or out of step with each other (phase), is decisive whether they will add together (in case of resonance) or cancel
[18] dependent on the data base characteristics they may also turn out to be of random nature

extent and other incomplete hypotheses in decisional processes, especially if the uncertainties are undescribable such as elastic bearings, degree of fixing, composite action between concrete and steel, crack formation and propagation, anchorage forces, dowel action, quality in construction or design and, if there is only partial or vague object information at disposal. Idealization errors in mechanical formulations come from inadequate sub-region size or connection/element type (missing shear deformation capability, crack development), insufficient mesh refinement (simplified discretization), interacting or missing model variables (aggregate type / size) and, from interaction of nonstructural components with the structure (Link, M.; 1998). Errors in statistical formulations come from parameter estimation or sample generation always hiding important characteristics of the original structure and so leading to an inadequate selection of the probability distribution and approximation functions.

In the same way as genetic algorithms, neuronal networks represent a black-box not allowing a verification and replication of results. Both rely on exact input data which, together with the resulting output data, are used to produce a model. Even if everything is tried to make best, predictions have to be limited within the ranges used in the training. This is not easy, since every structural system is different. Neuronal networks are conceptually different from natural fuzzy sets and so -if used-, one should apply them at quite different time scales to be beneficial for both fields. They might be used to minimize errors of fuzzified ideas by learning rules, or of imprecise quantities and relations. However, though the net might be rendered more flexible by fuzzifying not only the parameters themselves, but also the convergence evaluation criterion itself, their suitability in direct damage assessment is very questionable. The author doubts their efficiency since, even using them to select representative fuzzy sets, too many training samples would be required. Not only is the evaluation multimodal (unbounded number of different rules and/or membership functions having similar performance), but due to different performance with similar rules and membership functions, it may turn out to be deceptive (Shi, Y.; 1999).

3 Scheme of the Actual Evaluation Procedure

3.1 Selection of Damage Indices

Since failure modes cannot always be explicitly identified or directly detected, damage simulation has to be performed indirectly via the structural response. In contrast to strength-based damage indices response-based damage indices do not directly depend on the system / material itself or the type of loading. Nevertheless, both type of indicators should be simple and consistent. They should be conceptual, i.e. directly corresponding to a clear aspect of material or structural behaviour, directly measurable by practical experimentation, and not require extensive post-processing. They should inform not only collectively about all but also about one unique material property (Menetrey, Ph.; 1996 and Laermann, K. H.; 1996). Sensors should simultaneously be low-cost, lightweight, easily installed, highly sensitive to damage evolution, but insensitive/robust towards distortions (noise, wind, changes in the ambient conditions), indeed contradictory requirements!

In order to evaluate structural damage several damage indices are introduced. Whereas the indicators for local damage refer to the hysteretic characteristics of particular system details, those for global damage predominantly refer to changes in the structure's modal parameters. Both consider static, dynamic and impulsive characteristics of the structure and are derived from the available literature. In analytical damage models usually two classes of damage indices are involved. The first class refers to strength-based damage indices themselves not requiring analyses of the structural response, but only calibration against observed damage using a large database. The second class refers to response-based damage indices requiring analyses with respect to the structural response.

Response-based damage indices
Maximum deformation

- Drift ratio $D_i = \frac{u_{max}}{H}$ (interstory better than overall drift); u_{max} represents the maximum displacement of an inelastic system being normalized to the storey height; since damage to a structure after having been tested for a second time is more than damage after the first test, the drift ratio has to be complemented by other indices such as stiffness before and after loading; primarily considering the flexural response it informs about plastic rotations or distortions, but not about instability or repeated/cumulative load reversals, neglects shear force transfer or inelastic response overestimating damage of ductile structures (Skaerbaek, P. S.; 1996 and Köylüoglu, H.U. et altri 1997) (I-1)

- $D_i = 1 - \frac{max.deflection}{stat.deflection}$ (I-2)

- Displacement ductility $D_i = \mu_d = \frac{u_{max}}{u_{yield}}$ or $\frac{u_{max} - u_{yield}}{u_{failure} - u_{yield}}$; u_{max} represents the maximum displacement, while u_{yield} is equivalent to the yield displacement; the displacement ductility represents the most traditional damage indicator (I-3)

- Stiffness Degradation $D_i = \left(\frac{k_m}{k_f}\right)\left(\frac{k_m - k_0}{k_f - k_0}\right)$; k_f represents the secant stiffness at failure, while k_m represents stiffness at maximum deformation caused by the applied loads and k_0 the initial tangent stiffness, respectively; the index does not take into account damage caused by repeated load cycles (Williams, M.S.; 1997) (I-4)

- Stiffness ratio $D_i = \frac{failure\ M/\varphi}{first\ yield\ M/\varphi}$ (slope); describing loading and unloading it neglects damage propagation (safe ≅ 1%, collapse ≅ 20%, Bachmann, H.; 1999, Park, R.; 1996). It only serves for model adjustment or global damage identification (this does not apply for the 2. and 3. spatial derivatives such as moment or shear force (Liew, K.M. /Wang, Q.; 1998). (I-5)

- Flexural Damage rotation $D_i = \frac{k_0}{k_{max}}$; k_0 and k_{max} represent initial tangent and secant stiffness at max deformation, respectively, inform about softening, damage susceptibility or areas of probable hinge formation better than μ_d, but neglects repeated load reversals (Nagar, A; 1995, Bernal, D.; 1992, Ghobarah, A.; 1999). (I-6)

- SARCOF Damage Indicator $D_i = 1 - \frac{EI(t)}{EI_0}^{-0.5}$; EI_0 and $EI(t)$ represent bending stiffness of an undamaged and damaged beam, respectively indicates type and location of deformation, rebar bond/slip/pullout.(I-7)

- Max Softening Damage Index $D_i = 1 - \frac{T_{i,initial}}{T_{i,max}}$; $T_{i,initial}$ and $T_{i,max}$ for the eigenperiod of the undamaged and damaged structure, respectively (DiPasquale, E. and Cakmak, A.S; 1990) (I-8)

- Another softening index has been described with $D_i = 1 - \frac{E_d}{E_0}$; E_d represents the secant elastic modulus, while E_0 describes the initial elastic modulus (Kraetzig, W.B. et altri; 2000). (I-9)

Cumulative damage or cycling effects (Singhal, A.l et altri; 1996)

- $D = \sum \lambda D_i > maxD_i$ with $\lambda = \frac{E_i}{\sum E_i}$; here E_i represents the dissipated energy at spring i ($D > 0.4$ damage beyond repair) (I-10)

- Low-cycle fatigue $D_i \frac{-0.2}{M_y \mu_{rot}} + \frac{\theta_{fail}}{\theta_{max}}$ has been applied to seismic analyses of structures subjected to strong ground motion; μ_{rot} represents the rotation ductility and θ for hinge rotation (involves the entire response history, but neglects the maximum inelastic deformation). (I-11)

Maximum deformation and cumulative damage

- Park and Ang's Index: $D_i = \frac{u_{max}}{u_{yield}} + \sum E_i \frac{\beta_{Park} \approx 0.25}{N_{yield} \times u_{yield}} \int dE_i$; $\int dE_i$ represents the incremented dissipated energy with β for strength degradation; N_{yield} and $u_{yield,i}$ represent yield force and -deformation, respectively; the index combines a simple ductility-type term equivalent to the maximum deformation experienced during cyclic loading with energy absorption due to cumulative hysteretic energy dissipation (minor –0.1-0.2, moderate –0.2-0.5, severe –0.5-1.0 and collapse –>1, Könke, C.; 1999c). (I-12)
- Stephens' Extended Index $D_{stephen} = \left(\frac{\Delta d_{pt}}{\Delta d_{pf}} \right)^{\alpha}$ describing damage sustained during each cycle of response with Δd_{pt} for the positive change/increment in plastic deformation in cycle i and Δd_{pf} for the normalization factor; $-0{,}2 < \alpha < 2$ represents the fatigue exponent (Stephens, J.E. et altri; 1987); (I-13)
- $D_{fire} = \frac{area\ of\ hysteresis}{stress\ range}$ fire damage describing accumulated stress (I-14)
- In summary, the most consistent indicators of severe damage are considered to be displacement ductility (I-3), "stiffness degradation" (I-4) and "Park and Ang's Index" (I-12). Whereas tor static loading the indices I-1 to I-6 are applied, for dynamic loading the indices I-7 to I-14 are used.

3.2 Data Preparation and -Analysis

A continuous challenge is represented by the vagueness and incompleteness of the recorded data. Supplementary information may only partially minimize inherent uncertainties which therefore should be described using adequate algorithms that do not hide or increase existing errors. In order to improve the quality of the results and reduce model complexity, the recorded data have to be analyzed and transformed adequately using specified methods and techniques.

A great impetus has come from the rapid development of microprocessor-based instrumentation attached to an oscilloscope for multi-channel data acquisition, remote-automatic reading, signal processing/transmission, -modeling, -storing, and -interpretation. Transient digital recording allows sampling rates of one point per microsecond or faster and so, strain gages may also be applied to dynamic phenomena. On-line display of all relevant structural reactions as function of each loading stage and an adaptive program initiating modifications to the analytic on-going procedure are indispensable in avoiding damage. Research aims at a monitoring system storing on only that info useful in both database format and content. Therefore in addition to sensor fusion, the data have to be transformed by scaling, translation, filtering, synchronization, or by frequency modulation[19]. In bridge evaluation, e.g. radar signals are analyzed by performing the following steps (Maierhofer, C. et altri; 2000 and BAST; 1999):

- reduce complexity, convert and/or smoothe the incoming signals
- transform with respect to zero
- filter to remove the error (in time domain for a vertical radargram and in frequency/space domain for a horizontal radargram)
- improve signal-to-noise-ratio by suppressing the undesired signal spectrum
- scale to improve depth resolution

Splitting of excitation and response signals into their constituent and homogeneous components (segmentation) leads to more detailed results than simply comparing an analytical model with the real structure. Although the merits of time versus frequency domain estimation still is a subject of great debate, specialists prefer a separation of low and high frequency image information by transforming impulse functions into frequency functions. When inferring the structural condition from -by Fast Fourier Transform or wavelet theory- converted frequency domain data, all modes should be considered. The FFT is an appropriate digital time-compression technique to quickly find out the spectral density of a stationary signal capturing relatively flat responses (voids), noise or time delay in a transmission path. Random variables are constant in time and space, while random fields represent correlated fluctuations in space, and random processes correlated fluctuations in time (Bucher, C.; 1999). Sensor fusion or cross-diagnosis use multiple signatures to compensate the inevitable variation of measured data (for direct error measurement see Daponte, P. et altri; 1998).

Among the applicable mathematical algorithms hyperbolic interpolation (displacement), linear or quadratic interpolation (forces), differentiation (unstable), integration (with limited time-step Δt using iteration represents a stable procedure also for nonlinear structures) and dimensionality reduction (truncation of

[19] variation of instantaneous frequency/amplitude of carrier wave according to the transmitted signal

higher modes) are worth mentioning. In order to include a time description in fuzzy functions, differential equations have to be established. Statistical methods refer to first and second order reliability analyses, to MCS, path-integral methods, discriminant analysis (parameter grouping) and factor analysis. Time-series modeling such as Auto Regressive Moving Analysis uses special interpolation schemes to accurately model hysteretic behaviour (generates e.g. earthquake time-histories) without time delay.

Mathematical and statistical algorithms for data reduction or uncertainty consideration should respect the know-how of experienced engineers having a generalized view of the whole issue rather than only insight in some specific details. Realistic results may only be guaranteed if manipulation is kept to a minimum (maximum information with minimum distortion). Modal expansion or other error-prone numerical operations such as twice-differentiation, double-integration, multiplication or squaring should be avoided as far as possible. Despite the care taken by structural engineers, most catastrophic failures occur as a result of human error, rather than as a result of uncertainty in loading or strength. So, adjusting higher safety factors would not prevent the failures cited above. Additional research is required to develop strategies for error control. Additionally to conventional methods expert systems and neuro-fuzzy systems deserve considerable respect (Zimmermann, H.J. et altri; 1995).

3.3 Modeling and Model Updating

Modeling of a structure should be performed in a flexible and consistent way considering the peculiarities of existing civil structures. Since different combinations of assumptions together with all possible sub-choices create more than a million different models, engineering practice is to base decisions on intuition and experience. Few models are kept as simple as possible with respect to the needs of all applicants (functionality) and as exact as necessary to explain the observations (time dependency or other constraints with respect to the application). Contradictory or misguiding data and view scattering parameters have to be canceled. On the one hand, such an information reduction to its necessary components helps the expert gain a good overview and describe holiness thus easing measurement interpretation. On the other hand, undesired but inevitable loss of relevant data of different type (geometric, multimedial and verbal) requires their simultaneous recording to illustrate a different abstraction degree or range of values.

As a model represents the data only within a specified quality criterion (predefined class of external forces), it should be only used within a finite set of information. Nevertheless, before a selected model can be applied, it has to be updated with the help of specified tests. Verification aims at a complete knowledge base without bugs or technical errors (random deviations or random errors). Here, completeness in the physical sense requires accurate conceptualization, while mathematical completeness aims at a converging objective function in order to avoid wrong results. A question to be asked is “is the model built according to its specifications?” Validation refers to differences between numerical and experimental results and, if the knowledge base is correct (nonrandom deviations or systematic errors). Its goal is wholesome consistency of all the incorporated details in order to avoid system faults. By iteratively correlating the model with observed data it tries to eliminate parametric or measurement errors and ambiguities (viability errors). Questions to be asked are “is the model sensitive and functional enough?”, “does it actually fulfill the purpose for which it was intended?” or “what part of the system does the model attempt to copy?? When testing a model on samples their produced results should correspond to the computed ones -ensure that correct results are not just annomalies. However, it is impossible to guarantee for every rare event, but only to inform about the degree of confidence. Accreditation or evaluation implies a final confirmation by the end user that a model is adequate and reliable for a specific purpose. Questions to be asked are “is the model user friendly and not too cumbersome?” or ”is the model valuable so that it can be applied by others than the developers?”

In other words, the necessary reparative work may be summarized with (Miskell et altri; 1989): verify to show the system is built right, validate to show the right system was built and evaluate to show the usefulness of the system. Uniqueness and stability are needed, especially if the geometry or material is nonlinear or, if the resolution is insufficient. A problem with verification, validation and accreditation arises from the fact, that most of the available information is incomplete, vague and uncertain. Specifications often do not provide a precise criterion against which to test. Therefore special tools are needed to account for this unavoidable situation.

Numerical analysis results have to be evaluated by a comparison with reality. Uncertainty within the input data, processing, evaluations themselves and system models has to be captured completely by reflecting all its characteristics (Beer, M.; 2002). In other words, “there is nothing so wrong with the analysis as believing its answer”. There are so many unknowns, and so many factors bear an influence that in no case will a model be able to gather the wholesome complexity of a problem. An apparent ease of detail incorporation often leads to unwarranted confidence in the validity of specific results. Reliability can be optimized with the help of either parametric (Baeysian approach in probabilistic structural models) or non-parametric (Regression Analysis, Neuronal networks in possibilistic behavioural models) identification techniques. Both improve limit state functions applying a least-square, a root-mean square or a simplified difference error formulation (Pandey, P.C.; et altri; 1994, Castillo, E.; 1997 and Shi, Y; 1999):

$$\frac{1}{N}\sum\left(T_i - O_i\right)^2, \quad \sqrt{\frac{1}{N}\sum_{n=1}^{N}\left(T_i - O_i\right)^2} \quad \text{and} \quad \frac{1}{N}\sum_{n=1}^{N}\left|\left(T_i - O_i\right)\right| \qquad (49).$$

In case of less than 30 samples N has to be replaced by $N-1$ (Fleischer, D. 1988). For a target output of less than ten equally contributing results, the absolute error functions have to be replaced by relative error functions such as (Molas, G.L. et altri; 1995, Tamaka, H. et altri; 2000 and Shi,Y.; 1999):

$$\frac{\sqrt{\frac{1}{N}\sum_{n=1}^{N}\left(T_i - O_i\right)^2}}{O_i} \qquad (50)$$

MCS and the less time-consuming Latin Hybercube using Spearman correlation coefficients are examples for appropriate parametric techniques for system properties. They allow a direct determination of sensitivity factors within a compass of LSF, while regression analysis and gradient methods are suitable for loading properties (Bergmeister; 2000). Evolutionary[20] and Tikhonov-Phillips (if confidence of the prior values cannot be estimated) programming techniques as well produce samples within the entire range of random variables.

Different estimators applied to the same model, but written in different formulations, lead to different values of the same parameters. Therefore mutual coherence should be studied on accurate models with sufficient samples to determine phase response function in one or several frequencies, translator factor (at the frequency of zero), resolution and linearity (Ghaboussi, et altri; 1995). All filtering and estimation techniques are averaging procedures which do not allow dealing with local defects, but only with *global parameters* such as drift, displacements, modes and stiffness easily being influenced by changes at a faraway location (Natke, H.G. and Cempel, C.; 1997 and Liew, K.M., Wang, Q.; 1998). These should therefore be complemented by an evaluation of *local residuals* such as the 2. and 3. spatial derivatives of stiffness (e.g. moments, shear forces), curvature ductility, strength deterioration, inertia, flexibility, energy transmission, -amplification/attenuation and -dissipation. Less corrupted by noise or high nonlinearities, the latter may as well indicate damage location.

The Kalman-Filter is a recursive Baeysian approach being capable of estimating missed state measurements for both linear and nonlinear models, in the same way as the max. likelihood method, rarely converges in reliable results (Caspary; 1998, Soong, T.T.; 1998 and Kalman, 1960).

Conventional feed-forward networks such as „Boltzmann machine“, „Hopfield network“ and „Kohonen network“ applicable for the planning of new structures

[20] primitive approximation to neuronal networks to detect local optima based on natural biological genetics combining a Darwinian survival of the fittest with a randomized, yet structured information exchange among a population of artificial chromosomes (Yeh, Y. et altri; 1993)

apply units do not learn or give their output back to a previous layer. Neuronal networks, however, can model constructed facilities using information from visual inspections and experimental measurement (inverse problems). They are a nonparametric technique directly relating the parameters for input (difference between damaging event and usual loads) and output (structural response -e.g. corresponding stiffness change or failure probability) with the help of a specified algorithm. Several authors have already documented their use not only in pattern recognition, data analysis, linear or nonlinear control (Ghaboussi, J. et altri; 1991, Wu, X.1992, Elkordy, S.F. et altri; 1993, Yeh, Y.C.; et atlri; 1993, Tsou, P. et altri; 1994, Masri, S.F. et altri; 1996, Contursi, T. et altri; 1997, Mukherjee, A: 1997, Castillo, E.; 1997 and Leger, R. P. et altri; 1998). Due to their success experts decided to use them also in condition assessment of civil structures.

Having been originated in 1943 by McCulloch and Walter Pitts neuronal networks are a very robust tool simultaneously achieving rapid convergence, great conceptual clarity and applicational simplicity. Even if no or few data of vulnerable parts are available, they detect faults and their root cause, assign unknown objects to an appropriate class or predict future behaviour. With the help of parallel distributed processing, i.e. using more than one computer to simultaneously measure several parameters or the combination of data from different sources, neuronal networks shorten search processes and separate individual effects of the large number of variables which otherwise would have to be assessed individually (Pedrycz, W.; 1993 and Castillo, E.; 1997). They manipulate or generalize symbolic data from qualitative measurement with implicit or intuitive knowledge and produce correct responses even when presented with partially incorrect or contradictory stimuli (Pandey, P. C. et altri; 1995). Similar to the structure and information processing capabilities of the human brain they transmit signals along incoming weighted pathways with the help of an assembly of numerous highly interconnected neurons (see Figure 3-1).

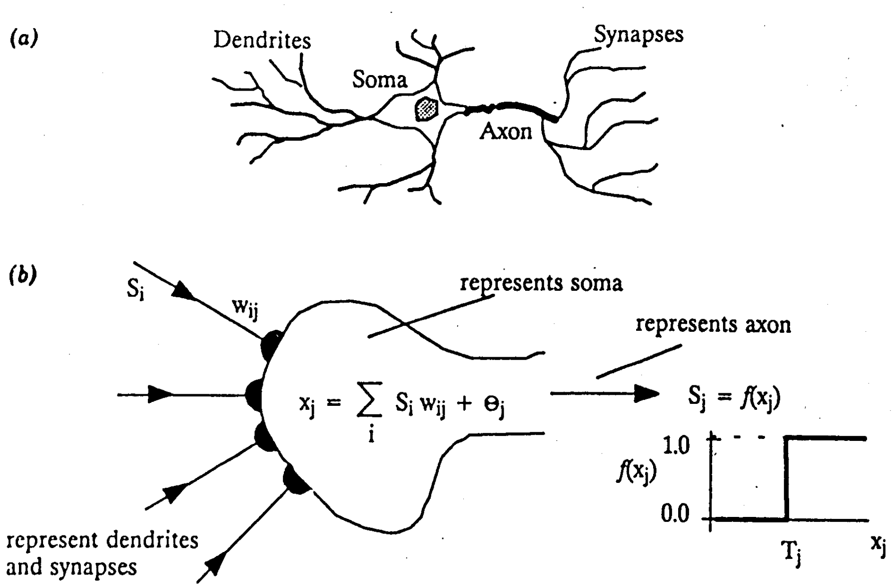


Figure 3-1 McCulloch-Pitts Model of Neuron (Wu, X. et altri; 1992)

Figure 3-2 illustrates the principles of network topology (Wu, X; 1992 / Ghaboussi, J. et altri.; 1995):

- set of parallel processing units connected in the form of a directed graph,
- no of units in input-, hidden- and output layers (depend on knowledge complexity),
- state of activation of processing units and their pattern of connectivity,
- employed propagation rule and activation function to compute output,
- type of training method and employed learning rule (creation of new units or extinction of existent connections between units, modification of weights, i.e. strength of relationship between the units along pathways, and application of neurons with different characteristics).

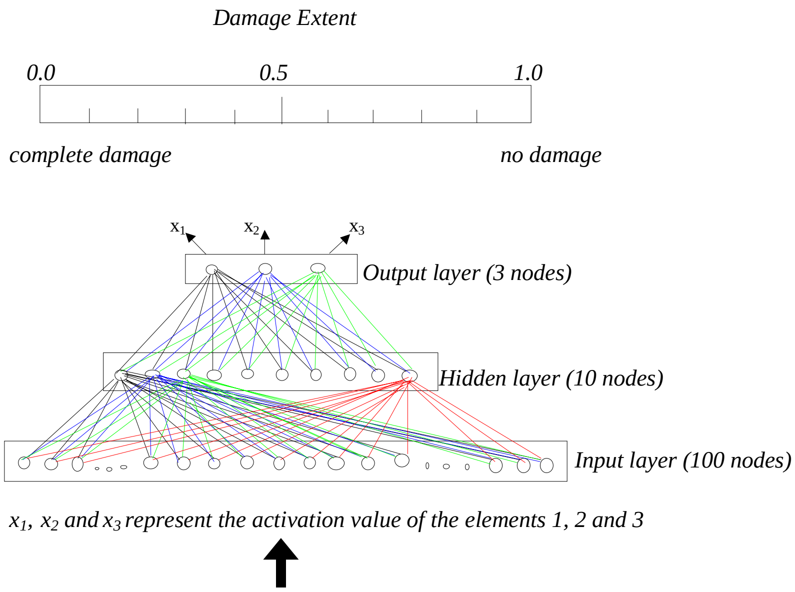


Figure 3-2: Application Sample for Damage Assessment

Instead of a simple least-square or root-mean square error formulation, they formulate the idea as a transfer sigmoid function and the performance as a fitness function (Leger, R. P. ete altri; 1998, Elkordy, M. F. et altri; 1993, P. Tsou et altri; 1994 and Ye, I. C. et altri; 1993). Each time the net is presented with a set of records (input and corresponding desired output), the information flows in a predetermined and self-organized manner from input to output layer (forward activation propagation) subtracting the calculated output from the desired response T_{neuron} (Molas, G. L. et altri; 1995). In the training process the error is back-propagated (from output to input layer) iteratively improving the connection weights until the discrepancy is minimized:

$$E^* = 0.5\left(\sum \left(T_{neuron} - O_{neuron}\right)^2\right) \quad \text{or} \quad E^* = \sum \left(T_{neuron} - O_{neuron}\right)^2 \tag{51}$$

$$O_{neuron} = \frac{1}{1 + e^{-\sum_{k=1}^{M} w_{ki} O_k}} \tag{52}$$

$$\left[\frac{1}{N}\sum_{i=1}^{N}\left(T_{neuron} - O_{neuron}\right)^2\right]^{0.5} \tag{53}$$

$$\Delta w = \frac{dE^*}{dw} = O_{neuron} I_i \left(1 - O_{neuron}\right)\left(T_{neuron} - O_{neuron}\right) \tag{54}$$

$$w_{new} = w_{old} + \Delta w \qquad (55).$$

Whereas supervised learning classifies new examples based on previous examples, unsupervised learning discovers characteristics beyond the information embodied in the training examples (Wu,X.; 1992, Takagi Hideyuki; 1993 and Ghaboussi, J. et altri; 1998). Rather than assigning random initial connection weights it has been found convenient to initialize models by pre-training it on a data set generated from numerical simulations. The relationship of network architecture with convergence and performance still is subject of current research (at present trial and error are used to determine dimensionality). Generally one hidden layer is sufficient for simple associations, if less than twice the number of nodes in the input layer are required. Having two hidden layers would allow prior data clustering of a high number of nodes and posterior further classification (Mukherjee, A. 1997).

An efficient training should constitute a „comprehensive“ data set of the undamaged and damaged structure, in case of need separately for all important failure modes or fault scenarios. Continuous and automatic incorporation of new nodes with new data dependent on the learning rate might avoid a repeated redesign of the net. If a training set only includes few values for a particular variable, it is best not to include it. With noisy data, even extensive training would not necessarily lead to a better performance, as then the net begins to learn noise impairing its ability to generalize. However, artificially generated data sets require a controlled addition of noise to simulate reality (Keger, R. P.; 1998 and Aktan, E.; et altri 1998). The more accurate the model used, the fewer are the training samples needed for adequate performance. Increasing the of neuron number improves training performance but affects testing performance. Even if tested with data not included in the training set (historical records), experimentally or analytically generated damage states, literature and expert opinion extracted from questionnaires are needed. Neuronal networks require all possible faults to be identified *before* training (Leger, R.P. et altri; 1998).

4 Fuzzy-Logic -for an Improved Condition Assessment

4.1 Introduction

In draft and design or conventional parameter identification the aim is to find out the structural response to induced loads directly proceeding from causes to effects. However, an evaluation of existing infrastructure has to be carried out without knowing their structural system. At best, it is possible to find construction plans or data having been produced during the construction work. Both consider only the initial situation neglecting later occurring events and the herein produced damage. Therefore at the moment experts perform condition assessment using subjectively selected damage indices such as I-1 to I-14 according to chapter 3.1 and even decide from a basis of isolated analysis results or pictures. The resulting drawbacks are obvious.

Extensive research has been performed on how to analytically conceptualize a structure directly from known loads and measured response data. In contrast to this direct[21] problem (see Figure 4-1) the process that defines causes from effects is referred to be inverse. Here, empirical, heuristic and approximate knowledge may guarantee serviceability, controllability and identifiability only, if measurement parameters are prepared or processed in a way that bias and variance errors are mitigated (Roca, P. et altri; 1997 and Natke, H. G. and Cempel, C.; 1997). Test and computer aided modeling applies knowledge from system identification, experimental design using safety factors and computer planning based on a set of simulations to consider the statistical variation of input parameters (see Figure 4-1):

Computational Planning (Inverse Problem)

Input	←	*System*	←	*Output*
Unknown		*Sought*		*Known*

Test Planning (Direct Problem)

Input	→	*System*	→	*Output*
Known		*Known*		*Sought*

Figure 4-1: Computational and Test Planning (Sandberg, G.; 1997)

When dealing with existing structures extreme loading and utilization conditions considering potential hazards dependent on the significance of the infra-

[21] in usual draft and design the structural response is defined dependent on a predefined, known system and the imposed loading, i.e. calculating effects from causes

structure, system vulnerability and environmental conditions (soil-structure interaction) have to be analyzed simultaneously. Structural safety and serviceability may only be guaranteed if technical aspects and user requirements are evaluated with respect to past utilization, equal utilization with different parameters, and modified utilization. At the moment a prevailing lack of knowledge about type/location of failure modes or potential damage processes often makes exact analyses difficult. Since a systematic procedure is not available, many problems are overseen and others overestimated. Besides, with the available theories never quite fitting the actual problem, there is rarely the chance to test a prototype as is the case in other engineering industry.

Experts tried to balance inevitable deficiencies by using more refined deterministic analysis methods. In order to deal with imprecise data of geometry, material, loading, boundary conditions and modeling they used tools based on probabilistics. Herein they were confident to receive reliable results, although the request of probability theory for a large data base is fulfilled only for possibly imposed loads or material properties, but not for structural reactions. Whereas in draft and design we dispose of an appropriate quality and quantity of data, this is not the case when evaluating existing structures. Since here, the available data are insufficient in quantity and quality, their stochastic character can hardly be valued so, that the probability of triggering events cannot reliably be established. Therefore the use of probabilistic methods has often proved to be unsuccessful in damage assessment (Bezdek, J.C. et altri; 1986, Hartmann, R. et altri; 1997, Starossek, U.; 1999, Schueller, G.J.; 1999, Schnellenbach-Held, M.; 2000 and Möller, B.; 2000). They were restricted to some selected types of system (e.g. nuclear power plants) and for few possible extreme events (e.g. earthquake, shock and impact). Also, there are few guidelines and those were only thoroughly investigated for single structural elements or subsystems risk analysis.

A very good solution of the problem was proposed in a recent publication providing a methodology especially developed for the Gerling insurance company (Meskouris, K. and Sadegh-Azar, H.; 2001). Potential structural damage dependent on location, value and vulnerability is revealed by calculating the latter in three steps. In level one (duration less than one hour) the expert aims at obtaining an overview of the structure considering its age, type of construction (number and height of stories, plan area, horizontal and vertical regularity, existence of soft-stories or short columns) and the soil conditions. The second level (duration less than six hours) aims at additional on-site measurements to reveal structural geometry, material properties, interstory drift, deformations and shear stresses in columns or shear wall. The third level comprises in-depth investigations with respect to the exact dimensions and mechanical characteristics. It may last several days or even weeks dependent on the specific requirements.

Neuronal networks may detect damage in general or discern it from noise, but as a black-box neither inform about the interrelation of connection weights or failure mechanisms nor discern between the relative damage amount among substructures (Elkordy, S.F. et altri; 1993, Masri, S.F. et altri; 1996, Mukherjee, A: 1997, and Castillo, E.; 1997). If one member out of many members is damaged while others are not, only a small change or a combined effect of all the damaged members is produced. So, in order to separate damaged from undamaged members, and the damaged members from each other, similar parameters would have to be grouped dependent on element type, material, continuity and geometry. This is possible only in very few cases.

Recently completed post-earthquake structural analyses of damaged buildings show that linear analysis alone is inadequate, and even nonlinear analysis is of limited use. The situation becomes worse, since current practice suffers from the shortage of experienced inspectors and the inevitable time delay related to in-depth analyses. Expert systems for various applications such as for the detection of surface damage at reinforced concrete elements have been developed, but none of them was able to cope with confusion or to consider uncertainty and vagueness adequately. Their suitability for general use is questioned, since each system produces answers that are different one from another (Hall, W.J; 2000).

Owners and stakeholders of civil infrastructure are not satisfied with existing subjectiveness, no matter its reasons. They fear a lack of safety and therefore require an objective advice on necessary monetary, material and personal expenses with respect to a future repair. The above mentioned problems show the strong need for further improvement. Due to its very special characteristics fuzzy-logic (mathematical tool to describe uncertainty inherent in human activities) allows the development of a calculation procedure which is optimally adapted to the problem to be solved. When analyzing features of relevant data, it can manage high dimensional search spaces which are too large for being captured with human eyes. Besides, capable of dealing with vague data, fuzzy-sets may quantify uncertainties of the cause-effect relationship of damage. The fact, that experts do not dispose of adequate results to accurately define it, or that they are not sure about their opinion, does no more represent a problem.

Although engineers tend to claim that every building is unique, many similar characteristics of civil structures should allow the development of type-specific condition assessment. This concept may be considered analogous to the manner in which we solve differential equations. The homogenous solution would be common, while the particular depends on boundary conditions considering the special characteristics of each case. Dependent on the selected hazard level and possibly occurring loads the analyst should select appropriate global and local damage indices on structural, element and material level. Whereas the global damage (structural level) gives a preliminary estimation of the total loss caused by e.g. an earthquake, the other two levels allow a more detailed review. Seismic analyses usually are performed using a beam-column system model on element level which, however, cannot describe the complicated nature of reinforced concrete such as non-rigid joints or bonding effects. On material level based damage indices include information about concrete (e.g. fracture indicator), about reinforcement (e.g. plasticity) and stress-wave propagation in general. Since severe damage primarily depends on deformation, indices related to ductility, drift ratio or stiffness ratio are more reliable than many of the apparently sophisticated indices considering the number of loading cycles. Reality idealization in a model only represents an incomplete description, but nevertheless promotes the clearing and development of new ideas by removing unimportant view scattering aspects. A validated model may serve for simulations verifying experimental results, but also for the design of a testing and instrumentation scheme with respect to additional analyses. In this way useless effort can be avoided and the expected range of parameters defined. Significant components for condition assessment are illustrated in Figure 4-2:

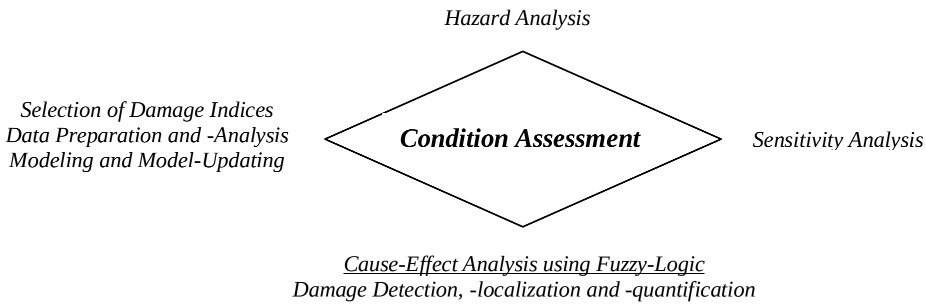


Figure 4-2: Significant Components for Condition Assessment

A holistic procedure composed of three subsequently performed steps, namely screening assessment, approximate evaluation and further investigations (see chapter 4.2.2, 4.2.3, 4.2.4) intends to classify structural elements and a whole facility by using trapezoidal fuzzy-membership functions according to chapter 4.2.1. Such a condition state assignment corresponds to the activity of vague modeling which is impossible applying of traditional methods.

4.2 Proposal for a Holistic Evaluation Procedure

As a consequence of the limited financial resources not all damaged facilities can be dealt with by applying the same accuracy. The author therefore proposes an evaluation procedure composed of three subsequently performed steps, namely screening assessment, approximate evaluation and further investigations for the case that one of the following facts have been realized (SIA 462; 1994):

- change in the type of using (different loads) or static system,
- obvious defects (big cracks, large deformations, substandard material),

- ground movements (settlement etc.),
- unusual events (flooding, wind, earthquake, fire, detonation of high explosives, bomb attack, vehicular collision, plane crash)

Such an evaluation procedure aims at assigning condition states with the help of trapezoidal fuzzy-membership functions according to section 4.2.1. In order to proceed in a very economic way, only those analyses appearing to be adequate in the actual situation are carried out. The "screening assessment" according to chapter 4.2.2 primarily uses visual inspection and especially selected material tests to determine simple damage indices such as interstory-drift, deformation, crack width, stress-level and loss of stability. It intends to eliminate a large majority of structures from an "approximate evaluation" and, to advise on immediate precautions with the aim of saving lives and high economic values. If the herein obtained information conforms the performed rough/simple calculations, one may explicitly quantify and locate the defects.

Otherwise, the "approximate evaluation" follows with in-depth geometry recording, material investigation and component analysis are performed. In order to avoid a random search of structural faults, a fault-tree may serve for systematic and objective investigations of relevant failure mechanisms dependent on the type of loading. Here the system and its failure modes are described looking if, also in case of a partial loss of key components, the required bearing capacity is provided after load redistribution (see chapter 4.2.3).

Special circumstances require "further investigations" using a fuzzy-algorithm to consider not only the actual material parameters, but also unaccounted reserves due to spatial or other secondary contributions (if, even with step two the weighted damage extent η cannot be defined explicitly, calculations based on fuzzy-sets help identify a vague or imprecisely defined relationship between damage cause and effect). In this way the facts are illustrated in a more realistic way than by using crisp values (chapter 4.2.4). Fuzzy-sets do not only serve for condition state assignment (chapter 4.2.1), but also for calculating risk p_i since the latter is a function of the weighted damage extent η (p_i additionally depends on the prognostic factor L and on the component importance I).

L_i describes the question whether an ill-defined event occurs or not and therefore is also apt to describe risk itself. Decisive is also the extent to which an event occurs considering imprecise characteristics or variability of failure. Therefore, when using a fault-tree the sum of the possibilistic values within a branch *is allowed to* exceed 1. Probabilistic values, however, are well-defined assuming that a failure mode with respect to a specific load path has already occurred, i.e. the likelihood of outcomes is relevant. Therefore their sum has to be below or equivalent to 1.

Again, the evaluation of risk comprises the estimation of the prognostic factor with respect to undesirable events L_i, their importance I_i and the expected weighted damage extent η (equation 62). Risk describes the degree of threat and is often expressed in terms of cost (Moser, K; 1997). The intention here is to estimate risk and not only a statistic failure probability $P_{f,stat}$ itself only representing the integral of a limit state function as is described in Figure 2-35 and only emphasizing one failure mode, namely that *one* representing the minimum of equation 38). The topology of the whole structural system has to be adequately considered e.g. with the help of formulas for element conjunction (serial system) and element disjunction (parallel system) according to section 4.3. Applied in matrix and vector calculations fuzzy-sets describe multi-state characteristics of the components (chapter 4.4).

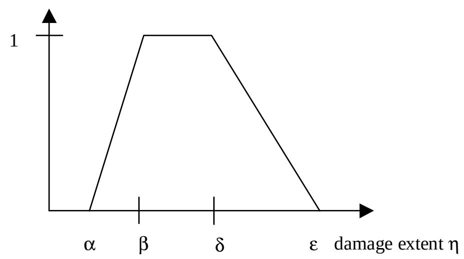
4.2.1 Fuzzy Membership Functions for Condition States Assignment

In 1964 Dr. Lotfy Zadeh (UC/Berkeley) extended a superset of conventional (Boolean) logic to handle the concept of partial-truth with linguistic expressions such as „high or large input variables produce medium output values“ and introduced fuzzy-logic as a mathematical tool. Up to now, it has already been used in conflict situations, decision making within strategic game theory, fabrication, portfolio selection, in the calculation of system reliability or safety and, in the optimization of design with the help of expert systems (Chao, R. and Ayub, B.; 1996, Möller, B.; 1996 and Schnellenbach-Held, M.; 2000). Being a representative tool within information theory and a complementary supplement to existing deterministic and stochastic techniques, it may account for uncertainty due to sparse and noisy measurements (approximate quantity), or due to human error (qualitative proposition).

Qualitative data are obtained cheaper than quantitative ones. Besides, undue complexity reduction and simplification is avoided, since fuzzy-logic is optimally adapted to the manner engineers tend to think and reason, who approximately summarize subjective information, professional wisdom and intuition. Since damage assessment is performed linguistically (with a minimum amount of lexical elasticity) and the reasoning process will therefore always be approximate, it is almost plausible to directly combine linguistic with numeric information. Human thinking involves a gradual transition from membership in one class to non-membership in the class rather than an abrupt change from one class to another. Zadeh intended to describe both, the gradual membership of a value to a group and imprecise reasoning: “the theory of fuzzy sets is basically a theory of graded concepts –a theory in which everything is a matter of degree or, to put it figuratively, everything has elasticity” (Zadeh in Zimmermann, H.J; 1985).

So, instead of concealing imprecise input data with crisp bounds *M = [0,1]*, fuzzy-sets with continuous intervals $M \in [0, 1]$ are applied to reflect the vagueness of soft results (Cheng, C.H. and Mon, D.L.; 1993, Shi, J. et altri, 1999, Pandey, P.C.; 1995). Illustrating the degree of preference or opinion between unreliability and reliability, they avoid pretending an accuracy in reality not being existent, misrepresentation or misinterpretation (Möller, B.; 1996). An addition of unknown patterns leads to an extension of the resulting answer in both directions, i.e. towards “non” and “destructive”. So, the fact that it could fit into more than one class leads to more uncertainty, just as it should. Figure 4-3 and 4-4 show trapezoidal membership functions $\mu(DE_i)$ applicable for the classification of structural components $\mathrm{E_i}$. Condition states $\mu_\eta(x)$ are described with the help of a function for the damage level *x* or performance (Tilli, T.; 1995 and Jendo, S. et altri; 1998):

$$\eta = \left[\eta_i ; \mu_\eta(\eta) = (\alpha,\beta,\delta,\varepsilon)\right] \text{ with } 0 < \mu_\eta(\eta) \leq 1 \qquad (56).$$



$$\mu(DE_i) = \begin{cases} 0, & \eta < \alpha \\ \dfrac{\eta-\alpha}{\beta-\alpha}, & \alpha \leq \eta \leq \beta \\ 1, & \beta \leq \eta \leq \delta \\ \dfrac{\varepsilon-\eta}{\varepsilon-\delta} & \delta \leq \eta \leq \varepsilon \\ 0, & \eta > \varepsilon \end{cases} \qquad (57)$$

Figure 4-3: Mathematical Description of Trapezoidal Membership Functions

In order to obtain trapezoidal fuzzy membership functions with respect to a structural element, information about the dominant failure mode is needed (use the procedure outlined in section 4.2.2 to 4.2.4). For example, if maximum deflection is relevant, the weighted damage extent η can be calculated dependent on the damage indices $D_i = 1 - \dfrac{max.deflection}{stat.deflection}$. Engineering experience is required allowing a decision with respect to the characteristics of the functions defined by α, β, δ and ε. For the condition state „considerable“ the values $\alpha = 60\,\%$, $\beta = 64\,\%$, $\delta = 74\,\%$ and $\varepsilon = 85\,\%$ have been established by the expert according to inspection results. In this special case a damage extent below 60 % or above 85 % will certainly not occur (that is, one may select the fuzzy membership value with $\mu = 0$). Besides, a damage extent of 62 % or 82 % may or may not occur (fuzzy membership value $\mu = 0.5$). And finally, the damage extent will certainly be in the range of 62 % and 74 % (fuzzy membership value $\mu = 1$).

For the case, that several failure mechanisms contribute to η, more than one function may exist. Dependent on the number of damage indices D_i and on the selected weight w_i, not only the weighted damage extent can be calculated with equation 77 or 78, but the expert can decide about α, β, δ and ε. The terms “none (N), slight (SL), small (SM), moderate (MD), considerable (CN), severe (SV), very severe (VS), destructive (DS)“ can e.g. be used (Jendo, S. et altri; 1998). So, does a reference condition state matrix S (DE_1, DE_2, DE_3, DE_4) = (SV, N, MD, MD) e.g. mean that S for DE_1 is classified “severe”, DE_2 as “none”, DE_3 and DE_4 as “moderate”. Figure 4-4 illustrates that one condition state can correspond two different membership values $\mu_{E_i}(\eta)$.

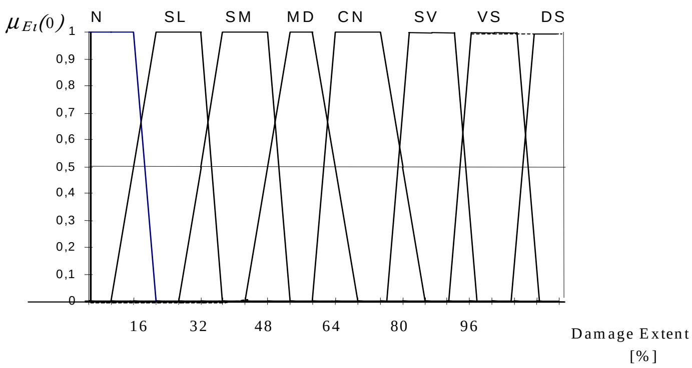


Figure 4-4: Fuzzy-Membership Functions DE_i for several Condition States

Here, e.g. a weighted damage extent of 80% belongs to a degree of $\mu_{E_i}(\eta) = 0.6$ to the condition state “considerable” and to a degree of $\mu_{E_i}(\eta) = 0.6$ to the condition state “severe”. It is again worth noting that in fuzzy-logic conflicting assignments do not represent a problem.

Condition states for safety, serviceability, reparability and aesthetics together with their manifestation can be assigned using Table 4-1 and Figure 4-5. For an assessment of the whole system instead of using eight linguistic variables, the author proposes to limit to five different linguistic variables, namely “very good, good, medium, emergency and collapse” being described in form of risk

with 0.1, 0.4, 0.7 and 1.0, respectively. Since risk analysis in most cases depends on an insufficient data base, a potential threat can only be expressed by approximate rather than by exact values. Calculated risk values may then be compared with predefined risk values according to Table 4-5 and 4-6 (see chapter 4.3). Insurance companies of world range and their experts dealing with structural analyses refer to two types of limit states. In general they discern between Serviceability Limit State (SLS $\cong 10^{-4}$ -out of 1000 similar components every 10 years one would fail) considering the effects of deflection or cracks by linear elastic theory and, the Ultimate Limit State (ULS $\cong 10^{-6}$ according to the nonlinear plastic theory -out of 10000 similar buildings every 100 years one would collapse).
The prestige numbers 1 to 9 reflect levels of the required quality of reinforced concrete components. So does the prestige number 1 accept large crack widths (0.6 to 0.7 mm) to be observed already from a short viewing distance, while the prestige number 9 rejects crack widths larger than ca. 0.15 mm. For crack widths exceeding a threshold value of 0.4 mm Jokela assumes that durability can no more be assured. Therefore, he nominates the vertical line parallel to the ordinate of the viewing distance (perpendicular to the abscissa of the actual crack widths) “limit of durability”, beyond which he recommends a rehabilitation with respect to an acceptable life-time (Jokela, J.; 1991).

Condition Index	*Manifestation*	*Possible Retrofit Measures*
1 –very good *Aesthetic effect (see Figure)* *Fully operational*	*Minimal deterioration evident (broken windows), isolated flexural or shear cracks (45 deg) at a width of <0.2mm, minor spalling*	*Immediate action not required; however, superficial repair (resurfacing, painting) is recommended as soon as possible to prevent further deterioration; provide adequate anchorage*
2 –good *Functional in most cases*	*Beginning of extensive flexural cracks, isolated diagonal shear cracks flatten towards 30 deg[22] (width[23] 0.2-1mm) only impairing durability, partial crushing in columns; nearly original stiffness and strength*	*Minor repair required; replacement of surface components,*
3 –medium *Serious deterioration in part* *Malfunction of the structure*	*Bi-diagonal shear cracks (width >2mm for 45 / 90 deg and >1mm for 30 deg), begin of debonding or rebar buckling, minor dislocation of key elements, major spalling in weak elements, local crushing of covered concrete without residual deformation*	*Although safety is not affected, an economic analysis of repair alternatives is recommended; the building partially is usable while being repaired*
4 –emergency *Significant structural (reduced cross-section) and nonstructural damage affecting load-bearing capacity* *Barely functional*	*Very large flexural or shear cracks, remarkable cover concrete spalled off, crush of concrete with exposed rebars due to shear and compression, disclosure of buckled rebars, separation of transversal rebars, partial loss of bond, disintegration of beam-column joints, visible residual vertical deformation of column/wall, column distortion and other dislocations, long-term reduction of structural strength due to corroded rebars and material properties affected, visible settlement and/or inclination of floor, lost most of ist original stiffness, but retained some of its strength, incompatibility with adjacent structures,*	*Immediate repair required, auxiliary structures and recalculations will be necessary, building is not usable until being strengthened;*
5 –collapse *major damage affecting overall load-bearing capacity*	*Broken bars, partial or total collapse of vertical bearing elements, major dislocations, stability endangered, lost most of its original stiffness and strength*	*Damage beyond repair requiring demolition; replacement of structural components may allow partial use*

Table 4-1: Condition Extent, Manifestation and Possible Retrofit Measures

[22] flat cracks are more damaging than very steep shear because they are opened up more quickly

[23] if the crack width is >0.2mm (prestressed concrete and concrete container) or >0.4 (normal reinforced concrete structures) and the concrete cover does not fulfil the predefined requirements

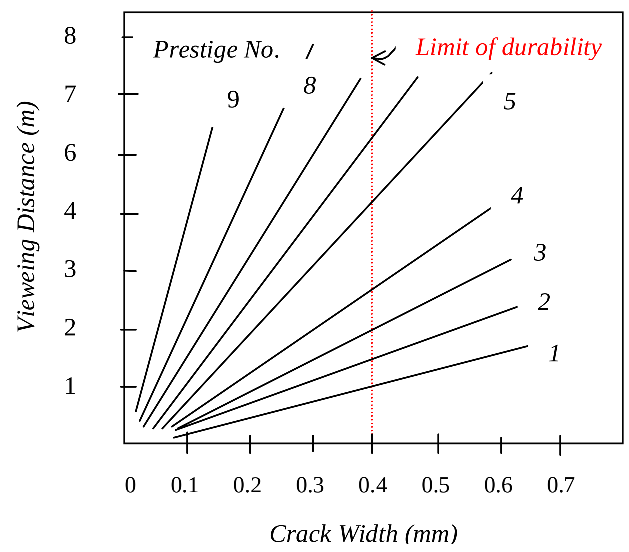


Figure 4-5: Aesthetically Acceptable Crack Widths (Jokela, J.; 1991)

4.2.2 Screening Assessment

Returning an old structure to its initial strength level with the same standard as a new one is inefficient with respect to the increasing difficulty of an assessment with increasing age. Nevertheless, it should be tested for obvious damage such as cracks, de-bonding, delamination (separation along a plane parallel to the surface) or section loss. In this way one may see, how far damage has progressed and, the velocity at which it may develop dependent on the type of usage. Normally, damage becomes obvious within the first 5 to 10 years, although potential weak areas may be hidden longer until a response quantity oversteps a threshold or, until many small damages accumulate into permanent damage.

The management of emergency services is vital for a rapid evacuation of occupants, securing medical treatment for the injured and reducing panic. During rescue operations, one may set about a previous rehabilitation procedure simultaneously assessing safety and usability. Whereas maintenance actions delay deterioration without improving a structure, repair improves it without altering deterioration rates. Information gathering bearing in mind the emotional relationship of the building represents a major part of the effort. Existing geotechnical reports on soil condition are just as important as design and construction documentation for the original and modified structure, but also for undamaged surrounding buildings of the same age. When reviewing existing analyses, one should assess the earlier work considering the design and fabrication philosophy of the relevant time by referring to old structural engineering textbooks. Maintenance or repair records inform about the buildings' history including previous condition and use. Old photographs or sketches are an integral part of data recording and invaluable for later reference. When newly photographing wide angle shots of the whole area should be taken first, so that the details can be located on the photo (see Figure 4-6 to 4-8).

Figure 4-6: Loma Prieta,California earthquake in October 17, 1989
http://geohazards.cr.usgs.gov/eq/html/damage.html

Figure 4-7: Loma Prieta,California earthquake in October 17, 1989
http://geohazards.cr.usgs.gov/eq/html/damage.html

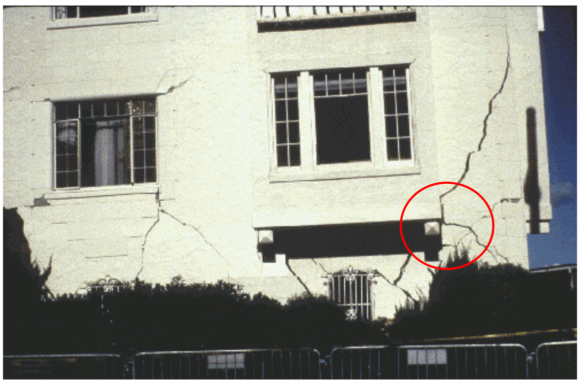


Figure 4-8: Loma Prieta Earthquake, San Francisco in October 1989
http://www.johnmartin.com/eqshow/647013_03.htm

Figure 4-9 shows the overview of the whole roof, while Figure 4-10 illustrates a detail of another roof part.

Figure 4-9: http://www.fema.gov/mit/bpat/bpn0299b.htm

Figure 4-10: http://www.fema.gov/mit/bpat/bpn0299b.htm

Then the produced detail photos can be taken to allow for further testing and measurement (see Figure 4-17 to 4-23 of chapter 4.2.4). A registration of their location help mark them on the wide angle shot. Video, 3-D visualizations and eyewitnesses allow reconstructing the chain of events and recreate the failure sequence. In this way type of stress or deterioration often become obvious.

Whereas the design of new constructions is regulated by official texts insuring uniformity, damage assessment implicitly requires the engineer to choose some level of safety and risk thus always leading to non-uniformity. The accuracy of an evaluation depends on the thoroughness of the inspection process, the evaluation methods used and the skill of the inspector. Older and more experienced engineers often dispose of pragmatic knowledge e.g. about a probable location of major deflections or about material behaviour (modern materials are more brittle, and so less resilient to movement). With all these requirements an inspector should not forget the constraints he is imposed to and intentions or expectations of the client thinking laterally as to how objectives can be achieved. One should consider who owns what, both on and in the vicinity of site, which obligations are related to (exploratory work may damage adjacent buildings and structures), and how the aspirations of the client and other interested party can be reconciled (Clancy; 1995). Here, nontechnical issues such as financial, legal, organizational, political and societal aspects have to be considered.

If possible, the building should be restored to its original state as soon as possible and with very little modifications, or it should be converted for an alternative purpose. A questionnaire cannot replace decisions but guide in searching the necessary information and in focusing the attention on the relevant issues. Even when being nervous it avoids him overlook important items and guarantees a uniform quantification of the observed condition. Despite the advances of nondestructive evaluation methods, visual inspection is still an integral part of condition assessment and the most widely used form. However, only a careful appraisal plan may avoid unnecessary testing and analyses. Prioritizing is no easy task if many facilities have been trapped.

On arriving at a building to be inspected, an external examination from a distance is advised to determine if it is safe to go further. A first guide to the likelihood of damage may be the degree of glass damage in the area or debris under foot. The safest path to take to a building minimizing the possibility of being struck by falling glass shards or masonry en route may be along the center of the street (Institution of Structural Engineers; 1996). Construction plans serve as a map facilitating safe access and egress. Once inside the building, numerous hazards may present themselves. Even if there are no signs of imminent structural collapse, engineers equipped with hard hats should remove concealing panels, damaged or dislodged cladding, glass and coping stone. Unsupported unreinforced chimneys have proven to be very hazardous.

Prior to a removal of the debris as much evidence as possible should carefully be recorded, since there may be slates embedded in furniture or walls being at risk of falling, unstable partitions and damaged or collapsed ceilings. Less common is the presence of hazardous materials such as asbestos disturbed by the blast, or spilled corrosive and toxic chemicals. One must constantly evaluate if it is safe to proceed further looking for possible hazards that could jeopardize structural performance. Vibration and noise are often characteristics of old service systems. Story drift, wall/column eccentricity, gaps between abutting construction, highly visible deflection, but also general deformation warn of incipient collapse or at least of moment redistribution. Movements, buckled columns, impact induced rotation or permanent distortion always lead to a loss of stability.

Flat slab constructions are always vulnerable, as they cannot behave in a ductile or dissipative manner. In case of need, punching shear tests inform about potential reserves. Since a building collapses only when it can no longer support its vertical loads, damage to the vertical load carrying system is far more serious than to the lateral. In a member properly designed for gravity loads no overstress will occur if the lateral loads are less than about a third of the total load. Nevertheless, a complete continuos stress path where the event energy may travel to reach foundations is required. Especially impulsive loading doses not only require sufficient strength, but also ductility of each element within the load path. Very important factors are, which member fails first and the combination of damaged members, since the limit state is defined by system failure rather than by locally prescribed limiting value of single elements. Additional loads induced on the building rest in consequence of a failed member require alternative paths of support. Therefore one should look for gaps such as discontinuous chords or defect connections incapable of delivering shear forces.

Redundancy is difficult to quantify or codify, but its presence may significantly improve ultimate performance. Fortunately most structures are considerably redundant, as their size or shapes of section do not strictly meet stress requirements, but satisfy deflection limits. Key elements designed with the „rule of thumb approach“ provide substantial built-in reserve capacity, provided that they are connected by vertical, horizontal and peripheral ties. Potential deficiency often is in the adequacy of the connections between pre-cast elements themselves being more vulnerable than the elements themselves. Thin proportions or sudden/abrupt changes in member dimensions (geometry), in stiffness or rigidity and in mass should be studied carefully, since yielding tends to concentrate at such discontinuities (see chapter 2.4).

Often the real bearing system is concealed by architectural finishes or non structural elements having been modified many times during their structural life or, even having assigned a part of the loads. Dependent on their material properties they may well absorb energy therefore requiring special attention. Different structural systems exhibit different degrees of vulnerability toward progressive failure. It might happen that the robustness or integrity is higher in a structure with a low degree of static indeterminacy. Although the latter often is described to correspond redundancy, the existence of hinges results to be more important during an extreme event. Moment-frame structures or braced frames with non load-bearing walls provide significant lateral stiffness, if detailed with bolted connections (welds are rigid and brittle, Zalka, K. A et altri; 1996, Stavros, A.; 1998 and FEMA-178: 1992). Concentrically or tension-only braced frames with infill or shear walls are less vulnerable than light-framed walls with shear panels or non-bracing elements (unreinforced masonry) only carrying vertical loads.

Basing a decision simply on the visible amount of damage is an incomplete strategy. Simple calculations should coincide with the observations obtained from material tests at critical locations. The ratio of cross-section areas of adequately reinforced walls and columns, to the total floor area above base informs about the most vulnerable building (Hassan, A.F. et altri; 1997). Type and geometry (area, position, length and width inclination, distance of a frame knot) of cracks indicate possible damage causes or failure modes, provided that only typical sections have been investigated. Figure 4-11 illustrates the development of cracks in a wall due to slab deflection.

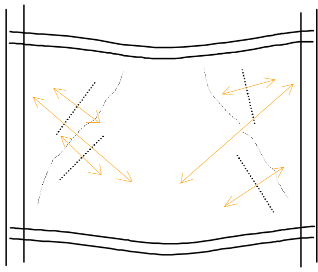


Figure 4-11:Development of Cracks in a Wall due to Slab Deflection

Concrete cover, presence of aggressive agents, carbonation depth, strength and corrosion tests of both, concrete and steal together with section loss in the most conspicuously damaged vertical and lateral bearing elements have to be verified also in a preliminary assessment. Reinforcing bar size and spacing can be easily documented using magnetic, radiographic or radar tools (see chapter 2.3).

It is of crucial importance to note any evidence of differential settlement and to inspect exposed foundation components for fracture (deep foundations such as piles perform better than spread footings). Base failure usually begins with the arching over the ground near a building. Figure 4-12 shows substantial cracking due to a clay inclusion.

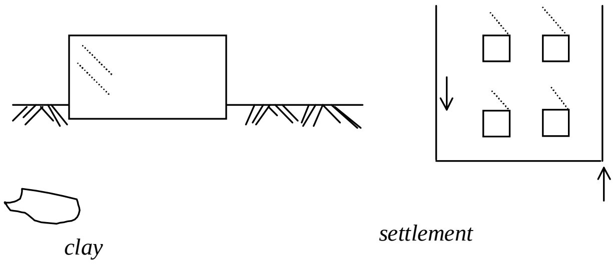


Figure 4-12: Settlement due to a Clay Inclusion

A thorough soil investigation with inspection pits, georadar or geotechnical borings usually is part of a detailed evaluation. The screening assessment shall only show if immediate precautions are required, if the system is intact or redundant and, if significant defects in key components can be defined explicitly (see Figure 4-13). If this is not possible, a detailed evaluation should follow.

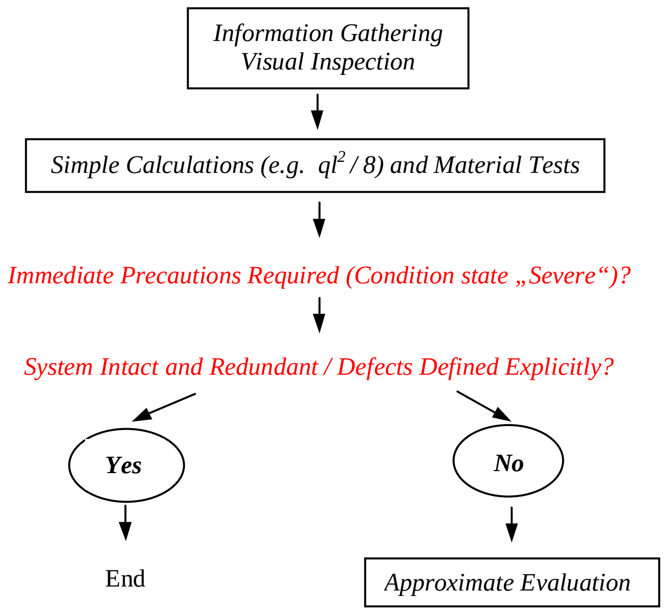


Figure 4-13: Flow-Chart for a Screening Assessment

4.2.3 Approximate Evaluation

Once, a structure is identified as a candidate for in-depth assessment, additional information has to be gathered in several areas. If there are no drawings of the building in plan and elevation/section revealing the intended dimensions, the exact geometry should be recorded using e.g. laser distometers together with a digital camera. A systematic search for damage is decisive for efficiently proceeding in an evaluation. It is appropriate to both, systematize defects with respect to their cause (selection of damage indices such as “maximum deflection”, “cyclic or cumulative effects” or a combination thereof according to section 3.1 and, systematize them with respect to the type of the structural elements. Existing reinforcement plans should be checked for conformance with respect to concrete composition and rebar type, quantity and location. Rebar deficiencies such as corrosion due to electrochemical deterioration are more critical than insufficient concrete cover or corrosion due to physicochemical disintegration. If the carbonatization depth or chloride presence are critical, the actual attack should be analyzed by exposing the reinforcement.

Discontinuities, i.e. a visible step at the member in cross section, are to be found where the load gradient is steep. Stiffness degradation and temporary hinge formation in important bearing elements should be investigated in detail differentiating between material and structural damage, often referred to as local and global stiffness. Adjoining buildings with different vibration periods can produce non-synchronized movement of adjacent exterior walls leading to out of plane impact forces. If their floors are at different elevations, they act like rams battering the columns of the neighbour building. The lower building <

50% tall acts as a base for the upper part of the higher building and by receiving an unexpected load suffers from the high stiffness discontinuity at its top level. Every approximate evaluation shall reveal structural behaviour in various conditions.

Frequently occurring component categories and their typical failure modes may be captured in a systematic and objective way by using a fault-tree which is optimal to verify hypotheses, illustrate fault sequences or failure paths dependent on the boundary conditions and find out a general interaction or correlation of several processes (see Figure 4-14). As becomes obvious the construction of a fault-tree requires professional expertise to formulate ranking alternatives in an adequate way. Only in this way potential failure modes can be discounted step by step to finally identify the decisive one, without exhaustively examining all the possible ones. Problems might arise if a fault-tree contains both, an event and its complementary event. Here, the analyst would obtain an erroneous result, since complementary and uncomplementary events would be treated as independent events though being dependent.

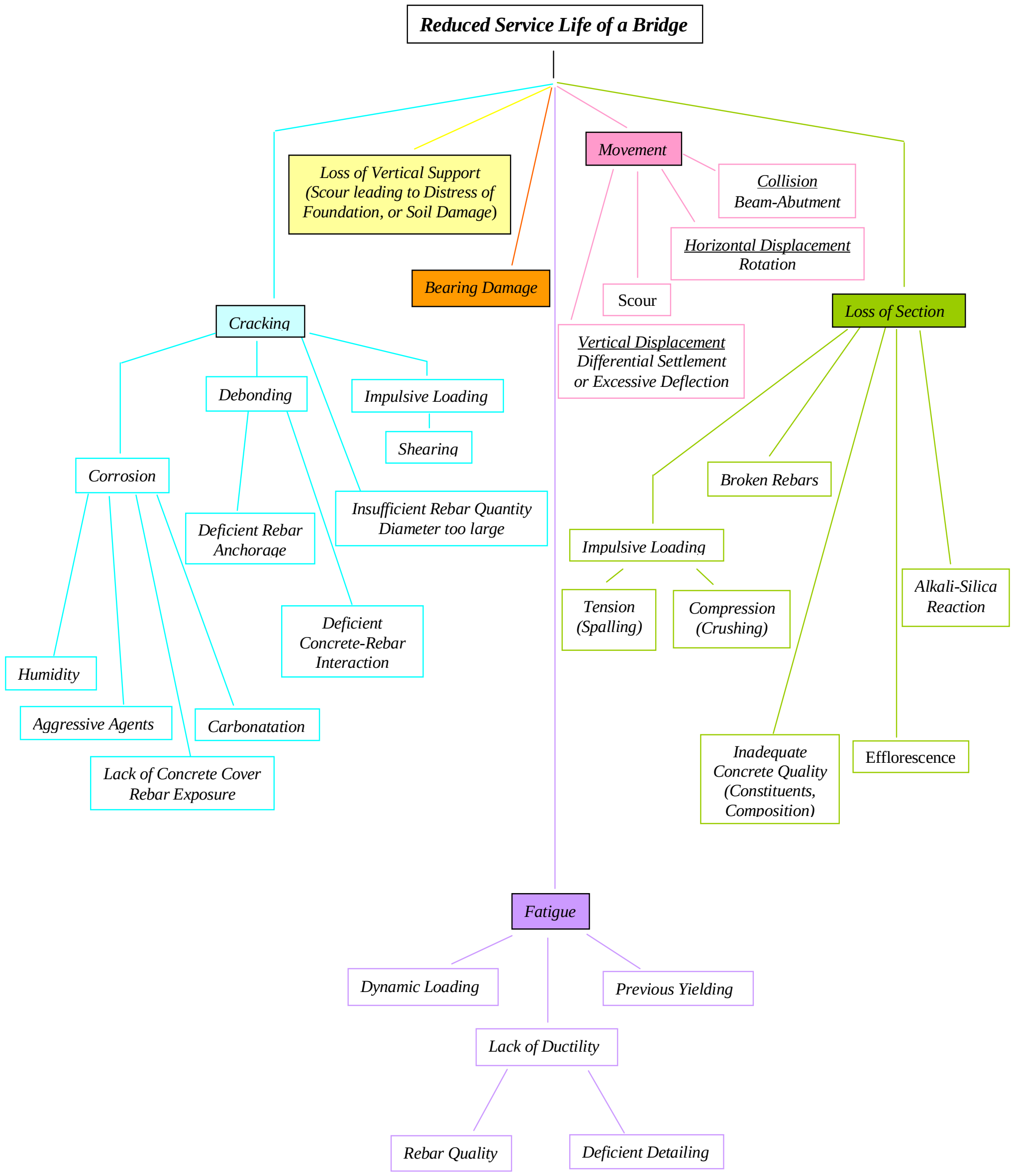


Figure 4-14: Example for a Fault Tree in Bridge Evaluation

Precisely defining potential consequences for any given scenario the fault tree indicates, where exposure is greatest by decomposing a complex structural system along its components, vulnerability and function. Problem size may thus be reduced by considering only a part of the structure replacing the residual structure by dynamic boundary conditions, or by describing the structure at two abstraction levels. In a single degree-of freedom model coupling effects (spring constants between the foundation, walls, floors, abutments, piers, beams and slabs) are synchronized in submodels whose qualitative attributes are deduced from visual inspection and quantitative parameters from in-situ or laboratory tests. In this way all important aspects for fault detection and reliability evaluations can be considered.

Explicitly defined parallel and sequential interconnections of all the possible failure mechanisms are constructed from the top-down, i.e. starting with a top event determining its causes and propagating new relations until all conclusions are reached (Khanna, P. et altri; 1992 and Sianipur, P.R.M; et altri; 1997). The quantitative evaluation is then carried out bottom-up, i.e. from intrusion to consequences along the path deciding about the importance I and the prognostic factor L (for a top-event the constraint with respect to the prognostic factor) may be defined with (see also chapter 4.3).

$$L_{top} = L_1 + L_2 + L_3 \geq 1 \qquad (58)$$

Usually damage is typical for a type of systems:

<u>Slabs</u>: vibration, loss of vertical support (step function shape with slab top displacement relative to edge support), curvature (central 1/3 to ½ span) with corresponding differential deflection (centerline minus near wall to slab length ratio) affecting the abutting walls, rotation of roof slab, crack aggravation over the support due to debonding, rebar exposure, bending or compression failure (crushing on the top side central 1/3 to ½ span), tension failure (scabbing on the bottom side central 1/3 to ½ span), punching shear in flat slab constructions (circle in plan view), direct shear,

<u>Beams</u>: loss of vertical support, deflection, bending or compression failure, rebar exposure or buckling, corrosion due to a loss of the alkaline protection, deformation, yielding, broken or pull-out bars may shifting a bending mode towards brittle shear or diagonal tension failure with a fracture of stirrups, crack aggravation over the support due to debonding and loss of concrete/rebar interaction (concrete teeth are zones between sets of cracks), bond splitting failure (slippage of the reinforcement over the supports due to inadequate anchorage of longitudinal or transfer reinforcement bars),

<u>Slab/wall connection</u>: loss of ductility, flexure at its top side (yielding or hinge formation at the centerline), curvature of the main slab, global rotation (max. wall displacement divided by the distance to the support), local rotation (displacement in hinge regions), concrete crushing, tensile cracks, loss of concrete/rebar interaction, deficient rebar anchorage at wall supports, insufficient load transfer (mutual and to the foundations),

<u>Precast elements</u>: embedded connections are locations of possible weakness (cracking causes a loosening), beam-column joints with eccentricities > 20%, deficient construction splices leading to hinge formation,

Shear walls: dependent on geometry, rebar percentage and loading a failure may occur in diagonal tension (large cracks inclined at roughly ½ -thickness from the wall face or 45° to the horizontal), in diagonal compression or, in sliding shear, out-of-plane cracking of end-walls, edge or boundary elements with inadequate reinforcing/confinement do not perform their function as a bracing, deficient shear transfer around openings ($\frac{h}{l} > 4$ might lead to overturning), buckling at the compression or tension edge if the lap of vertical bars was designed as a compression splice, inward top rotation,

Infill masonry walls and horizontal diaphragms: diagonal or stepped cracks if unreinforced, especially at large openings with overstresses due to arching between floors

Columns: inclination, p-delta effect, overturning, diagonal cracks inclined at roughly ½ -thickness from the face or 45° to the horizontal, concrete scabbing (depression due to removal of surface concrete -microstructure highly destroyed) at the rear face, cratering, spalling and crushing (excessive compression) at the front face, deficient rebar anchorage, rebar exposure/deformation/yielding, torsion or lateral torsional buckling (net of arbitrarily orientated inclined cracks), distress due to large shear wall openings requiring the remaining pillars/piers to carry all the shear, insufficient confinement or number of stirrups, interaction of short columns and strong infills due to a sudden change in stiffness, strong beam -weak column failure mechanism (see Figure 4-15),

Figure 4-15: Bidiagonal Shear Cracks and Spalling (Shear and Compression Failure) Alejandro Asfura/EQE International, Armenia 1999;

Moment-frames: excessive interstory drift (loss of verticality), inclination, "soft-story" failure (tall stories), insufficient bracing, stiffness discontinuity or degradation, buckling or lateral torsional buckling, long-term stress accumulation, ineffective stress relieving soft-joints, concrete spalling/scabbing, rebar exposure/deformation/yielding, ductile failure (bending)

Structural system: exceeding of acceleration limits (vibration), wholesome stability loss, accumulated dissipated energy indicating fatigue due to dynamic loading, uneven mass or stiffness for two orthogonal directions due to changes in member dimensions (an indicator is a change in eigenfrequency), insufficient bracing, lack of expansion joints, plastification of cross-sections, fatigue of materials becomes obvious by unstable crack propagation (consider length and width, inclination -vertical/horizontal, straight diagonal, steep, shallow curved),

Nonstructural elements: lack of adequate isolation, inadequate connections to structural elements (lack of stirrups or lateral rebars at joints), insufficient anchorage of mechanical and electrical equipment (computers),

Foundations: deterioration, material breakdown, cracking, corrosion, insufficient vertical rebars in the restraint area which cause sliding in the wall-foundation connection, inadequate strength of footings and/or piles and of their connections, bow perimeter walls with several basement levels, excessive earth pressure, distress related to ground movement, differential settlement, All the observed features should be documented in a pictorial presentation with additional detail photos of the critical area and wide angle shots of the whole area. An approximate evaluation should include an identification of the time-history and of all behavioural events that caused the observed condition by e.g. modifying features of single components and, through dependencies, those of the whole system. The type of loading (static, dynamic, impulsive or thermal) exert a strong influence on crack propagation and on the constitutive laws.

After having selected a desired hazard level identify possibly occurring loads, geometry and material considering the framing direction and lateral/vertical force transmission system, previous loading scenarios, boundary and environmental conditions. Precise definitions for local (one element) and global[24] (whole building) damage should be introduced, before weak points, appropriate measurands and possible/probable parameters together with reasonable tolerances of state parameters (upper and lower bounds) can be identified by using a fault-tree according to Figure 4-14.

If static loading is relevant damage indices such as ductility ratio, rotation ratio or curvature ratio should verify defects from overloading or load concentration (see damage indices no. I-1 to I-6). For dynamic loads fatigue, dynamic factors, resonance or seismic are relevant which can be verified using damage indices such as energy dissipation (see damage indices no. I-7 to I-14), while for impulsive loading concrete spalling or concrete scabbing combined with shear plug formation are obvious defects coming from large displacements or excessive deflection (see damage indice I-2). In case of explosion induced high-frequency ground motion the story drift (see damage index I-1) e.g. does not constitute an appropriate indicator, since vertical motion is larger than horizontal motion. The structural behaviour is therefore much different from that induced by an earthquake itself being characterized by low-frequency and low-peak magnitude (damage index “drift ratio” is appropriate). For both types of loading the stiffness ratio being dependent on the eigenfrequency change also represents a good indicator, since even very small (often not visible) cracks always produce stiffness degradation over the entire structure. The only problem is to infer strength from stiffness.

Failure modes frequently occurring in bridges, are low-cycle fatigue using e.g. the damage index I-11 (alternating plasticity with strain increments of changing signs which reduce stiffness, strength and ductility), shakedown (elastic response if the maximal applied load does not exceed the maximum load reached in the past load history) and incremental collapse (irreversible deformation after a large number of loading cycles changing from elastic to plastic). At critical locations these occur before the ultimate unidirectional capacity has been reached (S. Sirisak and P. Basu; 1998).

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[24] the expression “cumulative” often is used if damage result from a serious of events

For a positive[25] phase duration $t_d \geq T$, the structural response is referred to as being dynamic (earthquakes with a frequency < 5Hz) whose maximum deformation equals (designed for pressure loading).

$$x_{dynamic-max} = 2x_{stat} \tag{59}$$

Short impulse excitations (milliseconds), however, are of high-intensity and non-oscillatory (a-periodic). With a positive phase duration $< 0.1T$ their maximum deformation equals (designed for impulse loading).

$$x_{impulsive-max} = \frac{x_{stat}}{2}\omega t_d \tag{60}$$

Collision forces and impact from missiles or aircrafts, plane crash, dropped loads, and debris from the building itself or from adjoining buildings (5-10Hz) require an analysis with localized or concentrated loads, while accidental or weapon induced explosion and blast (> 10Hz) can be analyzed using distributed loads. The resulting maximum overpressure load to be applied to a structure can be estimated from the overpressure of the incident impact pressure wave with factor two in case of free explosion and with factor three in case of closed explosion. Attenuation depends on the characteristics of the propagating pressure wave interacting with the ground surface. In loose soils it is approximately proportional to the $(\text{radius})^3$. An air blast induced ground motion can far exceed a major earthquake's peak ground acceleration. For large overpressures with a long positive phase, the shock will penetrate some distance away into the ground thus damaging the subsurface and buried structures by reflection and diffraction. Extremely hot fragments might trigger secondary explosions or fire in other fuels and chemicals nearby. Possible consequences are flow after an impinging shock front or other nonlinearities.

Damage sustained during fire differs in important aspects and significant details from that sustained in a hurricane. Being of low-intensity, low-speed, long duration (seconds to minutes) and essentially oscillatory (periodic) in nature, fire causes local and global damage reducing the modulus of elasticity and strength. Fire safety mainly refers to passive fire protection[26], although an understanding of human behaviour can be more important (Connor, D.J.O; 1995 and Structural Engineers; 1996). Not only a prevention of collapse or excessive deflection is important, but also of cracks and fissures. Thermal expansion may

[25] negative phase of a blast wave starts behind the front where the pressure drops below atmospheric

[26] defined in terms of fire resistance time (up to 4 hours for some major structural members) it includes maintenance of the loadbearing capacity, integrity (control of transmission of hot gases, flames and smoke) and insulation or limitation of heat transfer (differential heating by convection or radiation may cause bowing)

lead to a loss of effective section or at least to temporary distortions. Damage at connections often is well away from the actual heat damaged zone.

Cracks at beam supports are not only caused by expansion as the temperature increases, but also due to restrained shrinkage during cooling from fire-fighting water. Fire damage is not a sole function of the maximum temperature or duration, but also depends on various other characteristics such as type of fuel, fire load, degree of ventilation to the fire compartment and fire-fighting operations. Spraying hot concrete with water after reaching the point of uniformity causes more reduction in stiffness than two hours extended duration of thermal exposure below 320°C (above 320°C the hydrides dehydrate and calcium hydroxide produces damage, Nassif, A.Y.; 1999). The thermal shock inflicted by water quenching and its reversed thermal profile produces significant concrete fracture due to its incompatible constituents.

If the modulus of elasticity (controlling displacements) or strength (controlling collapse) is reduced, conventional analysis tools have to be adequately modified. Mechanical properties refer to strength and stiffness dependent on aggregate type/size and cement-paste percentage. Consider a possible concrete degradation with disintegration and a breakdown of aggregate cohesion. In fire assessment one should additionally determine thermal properties referring to the coefficient of the expansion (differences between cement and aggregate lead to a spalling of the concrete cover and expose reinforcing steel), to the conductivity and to the specific heat capacity. Exposure time, availability of air needed for combustion and ventilation have also to be investigated (Al-Mutari, M. et altri; 1997). A few tests on undamaged surfaces with a Schmidt hammer as a "reference" should be complemented by an US survey to locate hollow areas or deep cracks.

While the space temperature reached can be roughly gauged by the discoloration of concrete, a better estimation of the actual condition of the material properties would be given by the study of the physic-chemical changes with the help of a differential thermal analysis. The color of debris burnt material (pink) can be relieved by extracting small cores. Whereas the tensile stress is reduced immediately during a fire (yielding), the compressive strength nearly remains constant up to 300°C (at 500°C loss of 50% -buckling). Columns having a large surface area initially extend in length, before shortening due to stiffness reduction. Though being exposed on both sides, slabs mainly suffer from underside heat convection.

An overview of possible extreme events will be given in Table 4-2:

Environmental Influence	Stability / Strength	Human Factors
Earthquake	Material Breakdown	Lack of Knowledge
Wind	Collapse during Erection	Poor Design
Flood (Water Pressure)	Collision / Impact	Lack of Communication
Fire	Fatigue	Poor Work Practice
Explosion	Progressive Collapse	Poor Safety Procedures

Table 4-2: Extreme Events

An "approximate evaluation" should inform, if the load-bearing capacity of the structure is sufficient also in case of a partial loss of key components. Decisive is to know, if their failure jeopardizes structural configuration or, if the required bearing capacity can be achieved by load-redistribution. In case of explicit knowledge about what to do, an assignment of condition states with the help of fuzzy-membership functions according to chapter 4.2.1 may follow. If, due to complex failure mechanisms an explicit decision is impossible, further investigations according to chapter 4.2.4 are required. Figure 4-16 shows illustrates the approximate evaluation:

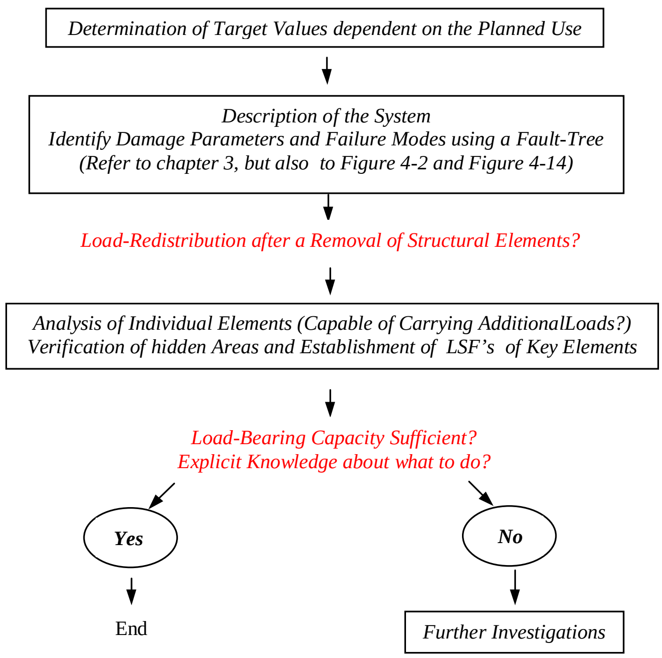

Figure 4-16: Flow-Chart for an Approximate Evaluation

4.2.4 Further Investigations

Special circumstances may require refined, but however cost-effective analyses. A structure should perform well in the required situations and not conform to any conceived solution. The engineer should therefore try to describe reality in such a way to effectively communicate risks, resource requirements and costs to the addressed decision makers. He should present relevant factors in the clearest possible terms and in an easily understood form, since the public needs to have a picture painted in order to finally comprehend the significance of essential activities and, thereby arrive at an appropriate option choice. Although additional material tests, statistical evaluation and load tests provide some additional information about unaccounted reserves due to spatial or other secondary contributions, a higher extent of risk has to be accepted with respect to both, safety and serviceability. In addition to technical aspects, as well interactions between legal, political and economical issues have to be considered.

Digital photogrammetric techniques or a laser scanner help create a 3D picture of the building simultaneously informing about displacements, settlement or crack geometry. In the close-range photogrammetry the determined coordinates may be related to an overall coordinate system by fixing a systematic grid on the facade of the building to refer to control points with known coordinates (Altan, M. O. et altri; 1998 and Streilein, A.; 1998).

Figure 4-17: Digital Photo of Damaged Schoolhouse obtained by KODAK DCS 200 (Plumb lines for Scale and Verticality Information), Turkey (Altan, M. O. et altri; 1998, Streilein, A.; 1998)

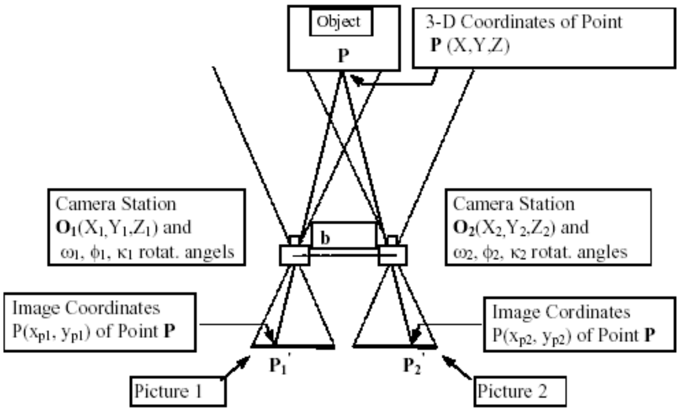

Figure 4-18: General Case of Photogrammic Data Aquisition (Altan, M. O. et altri; 1998)

A data model can cope with a large quantity of data by transferring them to powerful geo-information systems for storing and evaluation. While the camera selected for the images of Figure 4-17 and 4-19 was the KODAK DCS 200, processing and evaluation of the images was done with the photogrammetric software package PICTRAN. The result was the coordinates of the characteristic points defining movement of the building or crack geometry of critical components (see Figure 4-18, 4-20, 4-21 and 4-22). However, in order to use the above mentioned camera for photogrammetric records, deviations of the camera linse system from the orthogonal projection conditions have to be considered by the incorporation of some additional parameters, among them the inner orientation, scaling factor and the safety index (see Figure 4-23). These have to be previously defined in course of a field test calibration.

Figure 4-19: Detail Photo of Damaged Building (Volz, S.; 1997)

Figure 4-20: Incorporation of Image-Information for Computer Program (Volz, S.; 1997)

Figure 4-21: Overlays of CAD file of Cracks obtained by Photogrammetric Software and Image of Building (Volz, S.; 1997)

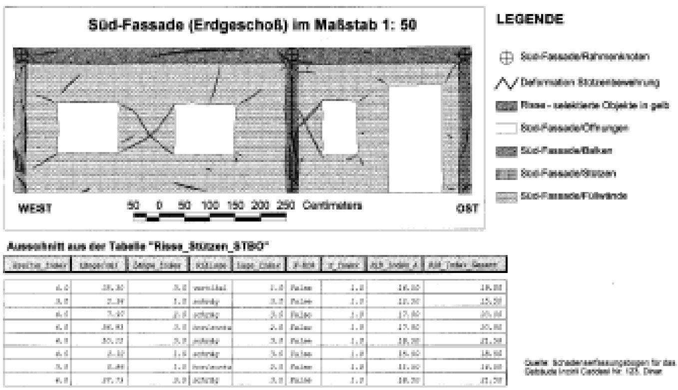

Figure 4-22: Automatic Selection of the Crack Structures using Example Data and the Result of the Indices (Volz, S.; 1997)

DISPLACEMENTS (mm) IN Z-DIRECTION OF THE FRONT FACADE

Axes / Story	1	2	3	4	5	6	7	8	9	10
3	744	241	-380	-605	-105	105	500	61	-152	-410
2	515	203	-237	-480	-133	133	347	23	-96	-273
1	217	130	-93	-254	-56	56	198	-17	-27	-155
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

DISPLACEMENTS (mm) IN Y-DIRECTION OF THE FRONT FACADE

Axes / Story	1	2	3	4	5	6	7	8	9	10
3	-205	-453	174	55	478	689	900	351	57	-16
2	-134	-392	183	58	389	555	720	222	22	-48
1	118	-386	257	72	258	351	443	127	09	-13
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

DISPLACEMENTS (mm) IN X-DIRECTION OF THE FRONT FACADE

Axes / Story	1	2	3	4	5	6	7	8	9	10
3	-54	44	82	269	61	-135	-208	26	362	101
2	-37	23	51	169	-12	-103	-181	89	372	128
1	-25	27	40	102	-72	-123	-174	91	376	-30
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Figure 4-23:Displacement Values of the Deformed Points in the Three Coordinate directions (Altan, M. O. et altri; 1998)

In order to exploit the load-bearing capacity to the utmost, all existing calculations have to be verified carefully or even replaced by new ones. Here, ductility plays a significant role, more than strength.

An event tree precisely defining potential consequences for any given scenario may illustrate, where exposure is greatest. It helps decompose a complex structural system along its components, vulnerability and function. With the help of hierarchical modeling, experts are required to explicitly define causes and effects, a potential interdependence or, which member is most sensitive to damage. Here, stiffness (rigidity), shear and elasticity modulus, compressive and tensile strength, friction coefficient and flexibility may indirectly be deduced from displacements, strain, sliding, percent change of mode shapes or other appropriate damage indices. Crack geometry and its distance to frame knot, concrete cover and deformed reinforcement bars together inform e.g., whether brittle shear or ductile bending is prevalent. In any case, visual and objective damage indices have to be reconciled by comparing the real structure with an adjusted model. This includes a verification of the local and global equilibrium, deformation or displacements at the support restraints, antimetry characteristics, strain and stress distribution. For example, by making a system alternatively stiffer and more flexible, one may bracket the data within upper and lower limits. More detailed results are obtained by splitting the signal in their constituents and homogenous components according to chapter 3.2.

Classifying structural elements and the whole system often is impossible, because the interaction between damage cause and effects is too complex. For this case an algorithm based on fuzzy-logic helps find out the dominant failure modes and their influence on the structural system dependent on their importance (see section 4.4). Additionally to an assignment of condition states according to section 4.2.1. Risk assessment may be carried out with respect to single components p_i and to the whole system P_f (see section 4.3). The latter are be finally compared with the acceptable values for risk according to Table 4-5 and Table 4-6, respectively. If, even a further investigation (see Figure 4-24) does not allow for a decision, load tests should be performed to clarify the ideas and estimate the existing damage extent together with risk.

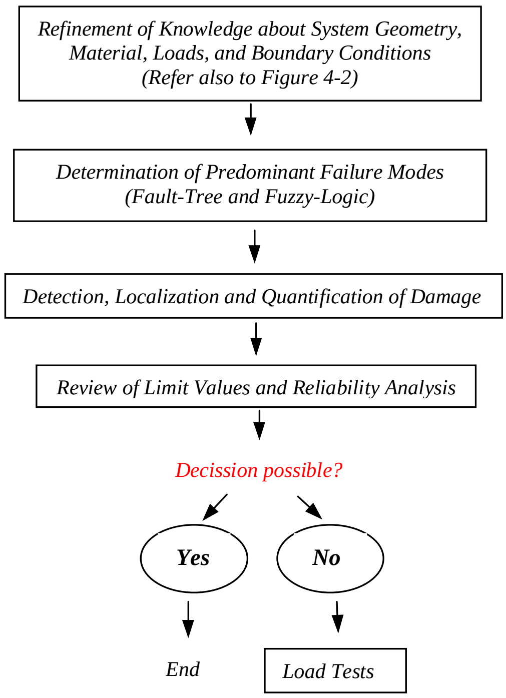

Figure 4-24: Flow-Chart for Further Investigations

For individuals and society it is important that both, assessment and remedial action justify a good return on investment. Therefore engineers should try to prove the efficiency of their activities. A decision on whether to adopt risk mitigation measures for loss reduction can be framed in a very straight-forward manner by comparing the cost of mitigation with the expected benefits (Kunreuther, H.; 2000):

$$E(B) = \sum^{T} \left(p_{old} - p_{new}\right)\left(L_{Mitigation}\right)\left(1+r\right)^{t} \qquad (61)$$

Here, p_{old} represents the annual probability of disaster causing a house to collapse without mitigation measure (p_{new} with mitigation measure). $L_{Mitigation}$, r and T represent the loss reduction from mitigation measures, the annual discount rate and the relevant time horizon, respectively.

It is worthwhile to equilibrate the relationship between expense and risk. However, above a certain limit risk does not decrease at the same rate as costs would increase with an applied measure, i.e. a rehabilitation is no more economical. Experts assume 1.5 times of expense to be equivalent to risk reduction by 50 %, but 3 times of expense to be equivalent to risk reduction by only 70 %. The relationship between expense and risk is illustrated in Figure 4-25.

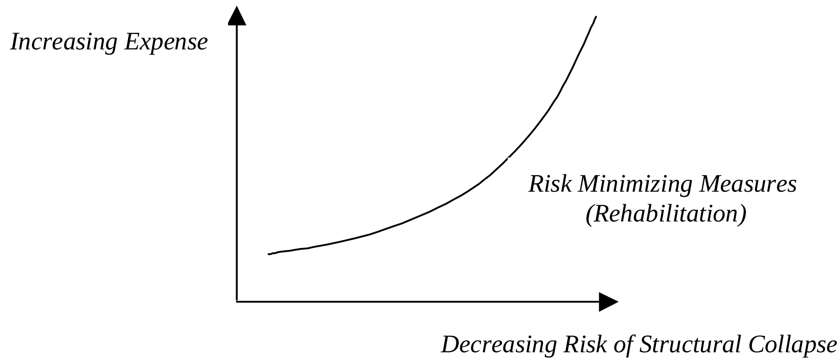

Figure 4-25: Cost Function for the Relationship between Expense and Risk

4.3 Determination of Risk

Postulating a limited set of expected failure modes is not always sufficient, since detectable signature changes are seldom directly attributable and every defect might -together with other unforeseen situations- become decisive. So, a determination of all possible scenarios to consider every imaginable influence would be required. Probabilistic simulations such as MCS do not necessarily produce the most significant data with respect to system behaviour, but only describe how the structural response might develop. Due to the interaction of many variables there is no simple and reliable way to predict which failure mode is dominant.

An important prerequisite in risk assessment therefore includes fault tree construction performed in course of the “approximate evaluation”. Equation 62 reflects the fact, that to each value risk being defined with $p_i \leq 1$, a degree of uncertainty is allocated. Risk is produced by a combination of various and ill-defined failure modes. Not only the “worst” case is considered, but all characteristics of system topology (determinate, indeterminate, redundant, stochastic dependent, stochastic independent). The weighted damage extent η is either directly defined, or determined by using equation 77. Risk also depends on the importance *I* of the structural component for the whole system and therefore is incorporated using an exponential function. In contrast to other mathematical functions the latter guarantees that for $0 < \eta < 1$ and $0 < L < 1$ one may conclude $0 < p_i < 1$ neither having an extremum nor nulls, i.e. is monotonically decreasing. Considering previously outlined facts risk for each structural component can be calculated with (Khanna, P. et altri; 1992).

$$p_i = \eta^I L \qquad (62)$$

L_i represents a prognostic factor[27] (does an ill-defined event occur?) and so is apt to describe risk itself (decisive is also its extent of damage considering imprecise characteristics or variability -therefore, the sum within a branch of the fault-tree *is allowed to* exceed 1). Probabilistic values, however, are well-defined assuming that a failure mode with respect to a specific load path has already occurred (relevant is the likelihood of outcomes. -therefore their sum has to be below or equivalent to 1). In summary, risk evaluation comprises the estimation of undesirable events using the prognostic factor L_i (degree of threat), the component importance I_i (Table 4-3 and 4-4) and the weighted damage extent η. Relevant are risk related costs and not the statistic failure probability $P_{f,stat}$ itself representing the integral of a limit state function (see see equation 38 and Figure 2-35) with respect to one failure mode, namely that *one* representing the minimum of equation 38 (Moser, K; 1997). L_i must not be confused with risk p_i and even less with $P_{f,stat}$ which represents a mathematically certain event obtained as often as required. The aim herein is not to describe the deviation or distance between expected and actual event (that would happen dependent on the prognostic factor).

[27] Although probability-, possibility- and fuzzy-set theory are not antagonistic and even have much in common, their differences still warrant a definite distincition. Actually, they have distinct roles in uncertainty description. Whereas possibilistic and fuzzy-set representations can capture weaker states of information, probabilistic tools cannot express ignorance (see chapter 2.6).

I = 1	*Significant for System Integrity as a Whole*
I = 0.9	*Absence would lead to Partial Collapse*
I = 0.8	*Nonfunction would largely affect Serviceability*
I = 0.6	*Nonfunction would slightly affect Serviceability*

Table 4-3: Component Importance

$L = 1$	*Failure is Certain*
0.9	*Very probable*
0.7	*Probable*
0.55	*Rather probable*
0.35	*Rather improbable*
0.1	*Improbable*
0	*Absolutely not*

Table 4-4: Prognostic Factor

Equation 62 shows, that different aspects of uncertainty, i.e. prognostic factor L and risk p_i, may be simultaneously treated. This kind of formula used by insurance companies does not work in case of very low values for the prognostic factor and high expected consequences. Since the risk attitude of people represents an important parameter, and bad feeling against disagreeable events significantly influences their attitude towards decisions, risk premium calculations of most insurance companies apply equation 63 to incorporate risk aversion.

$$\begin{aligned} Risk &= Hazard \times Vulnerability \times Value \times Aversion \\ Risk &= Hazard \times \quad Consequences \quad \times Aversion \end{aligned} \tag{63}$$

In economics risk aversion[28] represents a psychological factor meaning lack of tolerance towards a financial opportunity (risk averse consumers are less likely than risk-lovers to hold shares or risky financial assets). Here, it intuitively implies that when facing choices with comparable effective returns (on investment!), people tend to choose the less-risky alternative. So, if insurance compa-

[28] Although there is an intuitive appeal of risk aversion in conjunction with a stated preference for saving lives over money at the margin, one cannot logically infer the advisability of conservatism from these premises. There are risks associated with both overstating and understating risk.

nies are capable to improve their risk estimates and thus more accurately set their premiums, potential investors are more likely to supply capital.

In the same way as different failure modes have a sequence of interconnected causes, also a structural system and its risk P_f depend on both, the individual components' weighted state of condition, and the type of their configuration within the whole structural system. Without accurate data one cannot define η with certainty, because the cause-effect relationship of damage is not clear (refer to chapter 4.4). Old structures in general are redundant, i.e. dispose of sufficient key members with considerably high reserve capacity well replacing the function of failed components. Modern constructions, however, do not always dispose of an explicitly defined weak area. Various possible load and failure paths may exist at the same time. A simple addition of the values for p_i according to equation 66 would not always result in real risk of the whole system (represents an upper limit $P_f << 1$) but is calculated by a statistical combination. Therefore, the component risk values p_i are combined in a serial, parallel or hybrid manner according to equation 64 to 69 (Rosowsky, D.W.; 1997, Rackwitz, R.; 1997 and Schueller, G.I.; 1997a). In the same as is true for statistical "probability", also an addition of all partial risks would not result in total risk society is confronted with.

Considering only the actually weakest components in a deterministic sense is insufficient, since all other system components might change in a way that they become the weakest component. Besides, the individual strength of single elements does not necessarily prevent the overturning of a whole system which might be subjected to manifold partial failure events. So, for the statistical combination of risk with respect to one component is calculated with the equations 64 to 69. Large systems are therefore more vulnerable to global failure. The more ductile the components are, the better a structure can be approximated with a series system.

For a serial[29]-connected determinate or weakest-link system (as well static indeterminate systems in case of point[30] or section[31] failure) with a correlation coefficient > 0 (stochastic dependent elements) one may calculate

$$\max_{i=1}^{n} p_i < P_f < 1 - \prod_{i=1}^{n} (1 - p_i) \quad (64)$$

$P_f = P(g_1 \leq 0 \cup g_2 \leq 0 \cup .....g_n \leq 0)$ (Kraemer, U.; 1980, Ching-Hsue, C. et altri; 1993, Rackwitz; 1997, Rosowski, D.V.; 1997 and Fleischer, D.; 1988).

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[29] failure of one element implies system failure

[30] failure in one "point" is reached if isolated strain or stress values have reached a predefined limit with respect to both, safety and serviceability

[31] failure in a cross-section is reached if the maximum allowable internal forces have been exceeded

For a correlation coefficient < 0 the formula has to be replaced by

$$1-\prod_{i=1}^{n}\left(1-p_i\right)< P_f < \left(\sum_{i=1}^{n} p_i\right) \qquad (65)$$

Also, for $p_i << 1$ one may assume (Schueller, G.I; 1997a)

$$P_f \approx \sum_{i=1}^{n} p_i \qquad (66).$$

Parallel[32]-connected indeterminate or fail-safe systems (in case of system failure) with a correlation coefficient > 0 (stochastic dependent elements) are dealt with -here the load path is decisive (Kraemer, U.; 1980 and Rosowski, D.V.; 1997)

$$\prod_{i=1}^{n} p_i < P_f < \min_{i=1}^{n} p_i \quad \text{with} \quad P_f = P\left(g_1 \leq 0 \cap g_2 \leq 0 \cap .....g_n \leq 0\right) \qquad (67)$$

If the correlation coefficient is < 0 the equation is replaced by

$$0 < P_f < \prod_{i=1}^{n} p_i \qquad (68)$$

For example, if $n = 100$ stochastic independent components with an assumed component risk of $p = 10^{-4}$ are serial connected, the risk of the whole system would result in

$$P_f = 1-\left(1-10^{-4}\right)^{100} = 0.01 \qquad (69).$$

In case of a parallel connection, however, being more vulnerable to correlation, the risk of the whole system would result in

$$P_f = \left(10^{-4}\right)^{100} = 10^{-400} \qquad (70),$$

indeed negligible. Now, the calculated risk has to be compared with previously defined target indices (see Table 4-5 and 4-6).

In a realistic evaluation, several aspects have to be considered including (Institution of Structural Engineers, 1995 and Gavarini, C. et altri.; 1994):

[32] failure of one does not lead to collapse of the whole system

- integrity/consistency of the external and internal framework
- actual/incipient hazards (widespread collapse due to local failure)
- geotechnical risk (soil/foundation damage depends on topography)
- non structural risk (falling fragments from windows or other debris)
- external danger induced by surrounding buildings produce a domino effect
- indirect risk (shutting down of life-safety systems -light, water, ventilation, power cables)

At the moment there is neither a generally accepted method for dealing with diverse hazards, nor is there a comprehensive database to support a comparative risk assessment. Risk communication is confronted with severe difficulties due to the lack of an accepted and standardized risk language. Nonetheless, it is useful to develop risk values or reliability indices not only dependent on the likelihood of an unfavourable event, but also on its relative consequences in turn related to the vulnerability and importance of the structure (is failure slow enough to take remedial actions?) Whereas major damage and an extremely low likelihood of occurrence correspond to the risk category “very low”, small damage and medium likelihood of occurrence are categorized with “considerable”.

Risk is the natural consequence of uncertainty and so, is inherent in all human activities. Omniscience or omnipotence alone cannot reduce a risk to zero. On the one hand, correct information cannot avoid risk completely. On the other hand, incorrectness can drive risk to its maximum value. Insurance comnpanies of world range and firms specialized in damage assessment such as EQE International and thus estimating structural risk, argue that the terms risk and probability are interchangeable if the interpretations are unambiguous.

Slight damage to secondary structures is allowable, unless it would leave a detrimental effect on adjacent structures such as by secondary damage. Any constraint approximation function should differentiate individual and collective risk considering the voluntariness, degree of influence and societal attitude of the possible benefit. Subjective judgement is insufficient for decision-making, if the penalty for a mistake is severe. It is easier to design a structure that is safe than it is to calculate its risk with respect to one component or to the whole system. When defining acceptable risk values or reliability indices, it is appropriate to compare several well known risks in other human endeavors such as 10^{-6} for a train accident or 5×10^{-6} for death during a thunderbolt. Code provisions provide minimum values by using coefficients, but do not depict the hazard per se due to the lack of quality data of collapsed structures. Most estimates fall well below the background mortality risk level $10^{-5}/year/person$ (Ellingwood, B.; 1994).

In 1996 Menzies introduced a rating factor called Fatal Accident Rate also considering the consequences of failure. Provided that the relevant activity is of duration of more than 100 million hours, it is 0.002 for structural collapse, 1 for bus passengers, 30 for mountaineering, and 300 for motor-cycling (Könke, C.; 1999). The assurance companies propose to incorporate the psychological situation of human beings who e.g. consider a high damage extent with a low probability of occurrence to be worse than vice-verse (Hall, W. et altri; 2000, Ammann, W.; 2000 and Berz, G.; 2000). Ten people dying in one accident due to the failure of a public building is regarded to be worse than for ten people to die in ten accidents of private autos. “Low probability/high damage events” generally dispose of a relative small historical database therefore being associated with many technological and environmental risks. In case of traveling on an airplane, the passenger is voluntarily exposing himself to risk, while an employee working in a building that is supposedly “safe” is exposed involuntarily. So, different degrees of hazard are acceptable to different parts. Acceptable risk values also referring to the expense of accepted rehabilitation may e.g. be defined with Table 4-5 by using information from reliability indices for safety and serviceability according to Table 4-6.

0.0001-0	*Extremely Low Risk*
0.001-0.0001	*Low Risk*
0.01-0.001	*Medium Risk*
0.1-0.01	*High Risk*
1.0-0.1	*Extremely High Risk*

Table 4-5: Example for Acceptable Risk Values

	Target Indices of Reliability for the LS of Safety		
Expense of Accepted Rehabilitation	*High (brittle)*	*Moderate*	*Low (ductile)*
High	***3.8*** */ 3.7*	***3.3*** */ 3.3*	***2.8*** */ 3.1*
Moderate	***4.3*** */ 4.4*	***3.8*** */ 4.2*	***3.3*** */ 3.7*
Low	***4.8*** */ 4.7 (5.2)*	***4.3*** */ 4.4 (4.7)*	***3.8*** */ 4.2 (4.2)*
Expense of Accepted Rehabilitation	**Target Indices of Reliability for the LS of Serviceability**		
High	***1.0***		
Moderate	***1.5***		
Low	***2.0*** *(2.5 –low) (3.0 –moderate) (3.5 –high)*		

Table 4-6: Reliability Indices for Safety and Serviceability with Respect to the Expense of Rehabilitation (Fleischer, D; 1988, Wörner, J.; 2000, EC 1; 1999)

For the case that there are several possible failure modes and that it is not obvious, which of them dominates the structural response, an average value is calculated using the calculations based on fuzzy-logic as (see chapter 4.4).

4.4 Singletons for the Description of a Cause-Effect Relationship

An evaluation restricted to a fault-tree analysis is not always conclusive, e.g. in case of multi-state characteristics of structural elements. Collapse is always due to the compounding of several factors and, the transition between certain failure modes is not well defined but is characterized by qualitativeness. It is reasonable to assume that the overlap in interpretation among damage causes and damage effects, and the complex set of parameters that experts use to quantify condition states contribute to the vagueness of these linguistic terms. Since the number of combinations available for solving the problem is quite large and, each of the damage characteristic is conceptually possible, the required effort would be extensive. The number of combinations may be significantly reduced if, additionally to the fault-tree, calculations based on fuzzy-logic are applied to reflect a wide range of opinions. Together these tools may very well circumvent this combinatorial problem, in which most combinations would even have no real physical meaning. There are numerous reasons, why present condition assessment has not yet achieved the level of reliability and standardization required by the civil engineering community and by society. This is also due to the lack of verification, validation and evaluation rules currently being applied.

In order to explicitly determine component risk p_i, the cause-effect relationship of damage characteristics is described with the help of specified fuzzy-sets using the "Weighted Hamming Distance d_w" algorithm. Frustrated by the lack of fundamental reliability inherent in most computer applications, Richard Hamming dedicated his attention to automatic error minimization. He developed an encoding scheme capable of detecting the condition of any two bits, now known as Hamming Code and published his work "Error Detecting and Error Correcting Codes' in the Bell System Technical Journal Vol. 29, 1950, pp. 147-160. This mathematical code is designed in a way that, if a few errors occur, the right result can still be identified with the correct output. Even when being incorrect, the output is sufficiently similar to the correct result. Since the same applies for a vague notion of closeness, it may also very well serve in failure analyses. So, damage causes can be compared by measuring their dissimilarity using this distance measure capable of discriminating between similar characteristics.

Both, the recursive least-squares and the Hamming distance are consistent, reliable mathematical algorithms. However, the former is consistent only when dealing with data being on an equal footing (see chapter 3.3), while the latter is especially valuable for data pairs coming from different clusters. Although the recursive least-squares require minimal expertise to always converge in medium reliable results and therefore are applicable for general data analyses to optimize a weighted sum of squared errors, here the assumption and the problems to be solved are quite different. In contrast to conventional error minimization, we do not have only data from one pool. When analyzing damage mechanisms within structural components we are confronted with data from two different origins. Besides, the two groups of data, namely damage causes and effects, are characterized by different properties and therefore must not be mixed up with each other. So, there are several circumstances that mitigate the use of the least-squares approach for the purpose herein.

The weighted Hamming distance does not only allow detecting or correcting errors as was its original purpose, but also dealing with a combination of incompatible features with different scales. Here when comparing the importance of several damage causes with respect to damage effects, one may introduce a global criterion function measuring the extent to which the result can be regarded to be reliable. Whereas e.g. four damage causes and two or three damage effects may easily permit the expert to gain an overall impression of the structural condition, this is not the case for e.g. ten damage causes and eight damage effects. Since they all might interact with each other, the algorithm is a good tool to gain a view for the ensemble. When analyzing features of relevant data, it can manage high dimensional search spaces which are too large for being captured with human eyes. So, in order to reflect the real world containing hundreds of rules and dozens of variables, a combination of fault-tree analysis, fuzzy-logic and weighted Hamming distance are optimal.

Nevertheless, without affecting reliability some kind of simplification is appropriate when using the above mentioned tools. So, instead of parabolic, trapezoidal or triangular fuzzy-membership functions one may use single values, namely singletons. Having no left and right spread, but only one mean value, the latter are much easier in handling. As a fuzzy set with a membership function whose support is one single point in the universe of discourse being used if one gives up the possibility of summing degrees of uncertainties, but is restricted to comparing them (http://www.engineering.usu.edu/pclab/matlab-help/toolbox/fuzzy/fuzzyt 44.html and Dubois & Prade; 1988). Representing a numerical index of precision measuring specificity, it is the most precise values of a reference set for which the maximum possibility can be achieved in the sense of transformation from “possibility” into “probability”. So, even not allowing a detailed description of the microstructure itself, for the purpose of finding out the relevant failure mode, singletons may adequately express expert opinion. It has been shown in various investigations, that the form of fuzzy membership functions has only a small influence on the quality of the results.

Though the assignment of singletons per se is relatively subjective (they depend on the attitude of the expert), the proposed procedure guarantees that not already in the beginning certain failure modes are neglected just because their influence is considered to be low or, because only few information is available. As well in a later re-evaluation an examiner can replicate the train of thought and the obtained results. Possible deviations of attitude become obvious and therefore incorrect repercussions on condition assessment can be eliminated.

In order to specify the extent to which a potential damage cause V_i is related to a damage characteristic q_i (very weak, weak, medium, strong, very strong), to specify the correction factor K that a certain damage characteristic actually is exhibited ("very true", "true", "fairly true", "fairly false", "false" and "very false") and to compare several damage characteristics with each other, a specified fuzzy-algorithm is proposed herein. Whereas the „Relation Space" R uses the singletons μ_{ij}, K describes the situation with the singletons. The "Importance Space" H illustrates the relative importance h_{ij} of damage characteristics q_i / q_j such as "crack width is more important than its length or than its distance from a frame knot", "shear is worse than bending" with singletons h_{ij} (Ching, C.J. et altri; 1998). Note, that the relation matrix R, the importance matrix H and the correction vector K consist of singletons k_i being established by using data from visual, non-destructive and laboratory investigations interpreted with the help of expert knowledge or engineering experience. In R the horizontal axis (rows) represents the damage cause V_i, while the vertical axis (columns) summarizes possible damage characteristics or effects q_i. Both values have to be established using the results from geometry recording and form material investigation. These results are used as well to relate the damage effects q_i (weight them against each other) with respect to their importance. The weighting vector W^*, the weighted Hamming distance $d_w(V_i, V_j)$ and the confirmation degree C are calculated:

$$R = \begin{array}{cc} & \begin{array}{ccc} V_1 & \ldots\ldots & V_n \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ . \\ q_m \end{array} & \begin{bmatrix} \mu_{11} & . & . & \mu_{1n} \\ \mu_{21} & . & . & \mu_{2n} \\ . & . & . & . \\ \mu_{m1} & . & . & \mu_{mn} \end{bmatrix} \end{array} \quad \text{and} \quad K = (k_1, k_2 \ldots\ldots k_r) \tag{71}$$

The cause vectors can be described with

$$\begin{aligned} &V_1 = (\mu_{11} / \mu_{21} / \mu_{31} \ldots\ldots\ldots\mu_{m1}) \\ &V_2 = (\mu_{12} / \mu_{22} / \mu_{32} \ldots\ldots\ldots\mu_{m2}) \\ &. \\ &. \\ &V_n = (\mu_{1n} / \mu_{2n} / \mu_{3n} \ldots\ldots\ldots\mu_{mn}) \end{aligned} \qquad (72)$$

$$H = \begin{array}{c} \\ q_1 \\ q_2 \\ . \\ q_m \end{array} \begin{array}{c} \begin{array}{ccc} q_1 & \ldots.. & q_m \end{array} \\ \begin{bmatrix} h_{11} = 1 & . & . & h_{1m} \\ h_{21} & . & . & h_{2m} \\ . & . & . & . \\ h_{m1} & . & . & h_{mm} = 1 \end{bmatrix} \end{array} \qquad (73)$$

The damage effect q_i is h_{ij} times more important than damage effect q_j. The values of the rows in H are added up assuming $\sum_{i=1}^{r} w^*_i = 1$ and $w^*_i > 0$ in order to obtain the normalized “Weighting vector” W^* necessary to calculate the weighted Hamming distance d_w:

$$W^* = \left(\sum_{j=1}^{m} h_{1j}, \sum_{j=1}^{m} h_{2j}, \ldots, \sum_{j=1}^{m} h_{mj}, \right) = \left(w^*_1, w^*_2, \ldots w^*_m \right) \qquad (74)$$

The issue is, to find out which of the damage causes is mainly responsible for a damage effect. So, V_i and V_j of each fuzzy pair pattern are compared using the

Weighted Hamming[33] *distance* d_w

$$d_w(V_i, V_j) = \sum_{k=1}^{m} w^*_k \left[\mu_{ki} - \mu_{kj} \right] \qquad (75)$$

For $d_w(V_i, V_j) > 0$ one may assume that V_i represents the relevant damage cause. For $d_w(V_i, V_j) < 0$ one may assume that V_j is relevant. For $d_w(V_i, V_j) = 0$ both V_i and V_j are selected. So, each process step screens out one pattern identifying the other as the cause. The fact that positive and negative values unduly equalize, does not represent a problem, but is even desired (all aspects referring to the damage effects of V_i and V_j should be considered).

[33] An alternative is the Euclidian distance is $\sum_{i=1}^{n} w_k \left[\sum_{i=1}^{n} \left(\mu_{ij} - \mu_{ik} \right)^2 \right]^{0.5}$ (Tilli, T.; 1994)

By weighting the difference of the singletons μ_{V_i} and μ_{V_j} with the corresponding weighting values $w^*{}_k$ themselves calculated as a sum of the rows of matrix H, this is done with due respect.

It is worth mentioning that in the calculation process as a whole in spite of the apparently erroneous comparison of only two damage causes V_i and V_j, respectively, actually all singletons within the whole matrix are compared with respect to their significance. So is, e.g. in case of nine different damage causes V_i, the damage cause V_1 successively compared with V_2, V_3, ...to V_9 as long as $d_w(V_i, V_j) > 0$ (however, if the comparison of V_1 and V_2 leads to $d_w(V_i, V_j) < 0$ in the next steps only V_2 is compared to the rest, namely, V_3, V_4....to V_9. Again, if $d_w(V_i, V_j) < 0$ for V_1 and V_2, it has no sense to compare V_1 with all the other damage causes because V_2 has turned out to be more relevant than V_1). Thus it is guaranteed that each singleton is evaluated with respect to its "neighbours".

Dependent on the normalized weighting vector W^*, the correction factor $K = (k_1, k_2 ..... k_m)$, the degree of confirmation that a damage cause actually is relevant results in the confirmation degree of cause no. i:

$$C_i = \begin{pmatrix} w^*{}_1 \times k_1 \\ w^*{}_2 \times k_2 \\ \vdots \quad \vdots \\ w^*{}_n \times k_n \end{pmatrix} \times \begin{pmatrix} V_{i1} \\ V_{i2} \\ \vdots \\ V_{in} \end{pmatrix} \tag{76}$$

$$C_i = \sum_{j=1}^{s} (W^* \times K) \times V_i \text{, } i = 1, 2,..., n$$

From the general point of view a "confirmation degree" is appropriate to differentiate between trivial hypotheses. Here, the latter represent the selected damage cause V_i or V_j. Again to clarify the train of thought with respect to the proposed calculation procedure: the matrix for the Relation Space R, the matrix for the Importance Space H and the correction vector $K = (k_1, k_2 ..... k_m)$ have to be established by using data from visual, non-destructive and laboratory investigation themselves being interpreted with the help of expert knowledge or engineering experience. The normalized "Weighting Vector" W^*, the weighted Hamming distance $d_w(V_i, V_j)$ and the Confirmation degree C_i finally have to be calculated using the equations 74, 75 and 76, respectively.

In human evaluations there is always some fuzziness which has to be explicitly considered. First, the results of geometry recording and material investigation can differ. Second, due to a different background or interpretation of ambiguous linguistic phrases experts express differences in opinion. In course of the screening assessment as much information as possible is gathered and the structure visually inspected. Unfortunately often there are no maintenance or repair records and so, no data can be obtained about the history of the structure (previous condition and use). First, the whole structure has to be tested for obvious defects such as cracks, de-bonding, de-lamination and section loss by using knowledge from theoretical basics of damage analysis. Thus it becomes clear how far damage has progressed. Material investigation also informs about strength of concrete and steal, about corrosion and carbonization depth.

The differences between two attitudes with respect to damage causes and/or damage effects (low, medium, rather high, high) may be large from the qualitative point of view, but the actual differences in quantitative measures are small (e.g. few fractures of a mm in deformation). Also, even by a careful choice of fuzzy-values the meaning can be unduly changed. Some experts are hesitant or confused in their response because specific information on structural parameters or serviceability was not given to them. Having only a photograph they rely on their own intuitive knowledge. So, before continuing with the calculations it is recommended to evaluate the robustness of the applied fuzzy algorithm and to check if the obtained results are correct. This can be done for example with the help of parameter studies which inform about the effect different parameters exert on the calculated results.

Some of the formulas have already been explained for the identification of lifetime related crack causes such as alkali-silica reaction (Ching-Ju, C. eta altri; 1998). This publication, however, illustrates how to describe every cause-effect relationship, how to define the dominant failure mode, how to identify the weighted damage index and, how to integrate the calculated result in a holistic evaluation process. Actually, the obtained fuzzy values have to be retranslated in linguistic values in order to take back to approximate consequences of the original premises. This defuzzification process is necessary to make clear statements ending up in damage indices D_i to be assigned weights w_i according to their local influence on component or system performance.

In contrast to trapezoidal or triangular membership functions often being defuzzified with the center of gravity method, the results obtained from a calculation with singletons can be directly translated into human language, since the confirmation degree C already represents a kind of understandable solution. C does not guarantee an explicit information such as, that the damage cause V_i or V_j alone is decisive. However, it represents a degree of confidence as is already expressed by its name. So, the question “is the result sound?” is especially important if the analyst disposes of less expertise and incapable to judge the accuracy of an advice. The latter should participate in a weighted vote to come to an overall conclusion, where the weight is obtained heuristically. A low confirmation degree should be justified by additional information, especially if there are symptoms of unresolved controversies. Some failure modes may have more severe consequences than others.

Instead of the usually applied integrated damage indicator $D_{max} = max\{D_i\}$ the weighted damage extent η according to equation 77 represents a better damage measure, since it does not only include all individual damage indices (see chapter 3.1 illustrating the damage indices I-1 to I-14) together with their weights (significant participants are emphasized using weights w_i according to the opinion of the decision maker), but additionally emphasizes significant participants by applying them in the squared value. So, is e.g. “joint disruption worse than beam hinging”. η can be calculated dependent on all failure modes, i.e. dependent on the influence of the damage causes V_i on the structure being deduced from the weighted Hamming distance d_w.

When defining a crack-index it is not sufficient to include only information about the crack-geometry itself such as length, width, inclination etc. Engineers are required to record the location and number of the relevant cracks within the structural component. It is decisive if a crack is near a frame knot or if it is some distance away from it. A simple consideration of attributes cannot reveal importance and therefore, special analysis are indispensable.

Type, significance and reliability or truth of the obtained information may additionally be considered by dividing D_i with coefficients γ_i (see Table 4-7). The actual computation process will be shown later in the example problems (three span girder bridge and protective structure).

$$\eta = \frac{(D_i / \gamma)^2 \times w_i}{\sum D_i^2} \qquad (77)$$

Type and significance of the information		*Reliability of information* γ_i	
		Moderate	*Good*
Static calculations have been verified	*In part*	*0.3*	*0.4*
	Complete	*0.8*	*1.0*
Material tests have to be performed	*In part*	*0.3*	*0.4*
	In detail	*0.8*	*1.0*
Warning of failure	*No Cracks or deformation*	*0.4*	*0.6*
	Cracks or deformation	*0.6*	*0.8*
	Cracks and deformation	*1.0*	*1.0*
Influence on nonstructural components	*Negligible*	*1.0*	*1.0*
	Important	*0.5*	*0.7*

Table 4-7: Coefficients depend on Information Type, Significance, Reliability

For the case that the coefficients γ_i can be assumed with the value *1* the formula may be simplified to

$$D_w = \frac{D_i^2 w_i}{\sum D_i^2} \qquad (78)$$

5 Illustration of the Proposed Methodology with Examples

80 % of the existing damage reports have been based on pictures combined with questionnaires (FEMA, EQE-International, Loss Adjusters engaged after the earthquakes in Armenia and Turkey). The analyses have been performed "from a general point of view without developing mathematical models or simulating the failure process". Additionally, the captured information "is not at the level of detail needed". In some very special cases and due to clients or lawyers request detailed analyses have been performed. Since, most of those studies are confidential and cannot be distributed or published the applicability and reliability of the proposed fuzzy-algorithm are illustrated using a well thought theoretical example which, at the same time, guarantees the freedom to examine the sensitivity of the methodology respect to the parameters involved (Asfura, A.; 2001).

Figure 5-1: Highwaybridge Aschaffenburg
http://www.arminwitt.de/schreck.htm

Figure 5-2: Highwaybridge in USA (Internet-Newsgroup)

Even reinsurance companies of world-range do not gather the entire data base of civil infrastructure to be assessed. In the same way as numerous national and international universities they only "investigate some few selected aspects". In this chapter the application of the fuzzy-algorithm as part of further investigations (see 4.2.4) will be illustrated using two theoretical examples for reinforced concrete facilities, namely a bridge and a protective / blast-resistant structure, whose failure cause is not always as obvious as in Figure 5-1 and 5-2.

The bridge represents a well-defined line structure on which the train of thought may be better explained than e.g. on a complex 3D system of a building. For the second example a protective structure will be selected. Its structural behaviour usually characterized by rapid rates of loading/strain (impulsive loading conditions) -in turn leading to an increase of the material strengths of steal and of concrete- is more difficult to assess than an ordinary facility subjected to static or dynamic loading conditions.

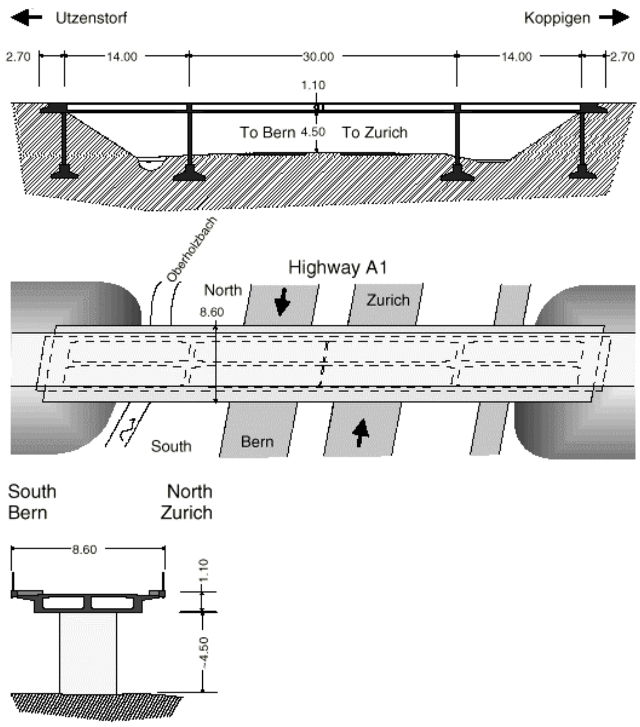

Figure 5-3: http://www.kuleuven.ac.be/bwm/SIMCES.htm

With respect to the first example (see Figure 5-3) it is assumed, that neither the screening analysis nor the approximate evaluation of the selected **three span bridge** has revealed the information required for a repair plan.

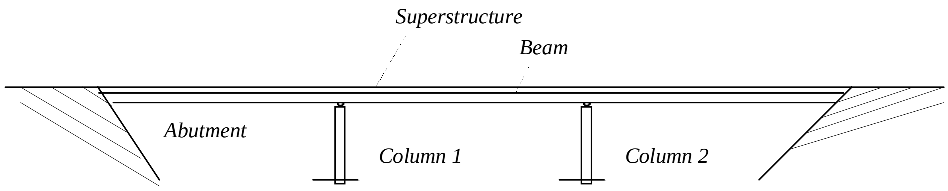

Figure 5-4: Simplified Structural System

In course of the screening analysis (see Figure 5-5) as much information as possible is gathered and the structure visually inspected. Unfortunately there are no maintenance or repair records and so, no data can be obtained about the history of the bridge including its previous condition and use. The whole structure is tested for obvious defects such as cracks, de-bonding, de-lamination and section loss. Thus it becomes clear how far damage has progressed. Material investigation also informs about strength of concrete and steal, about corrosion and carbonatization depth. It has been found out, that no immediate precautions are required even so the bridge is no more intact and redundant. The problem is, however, that the defects cannot be defined explicitly therefore requiring an approximate evaluation (see Figure 5-5).

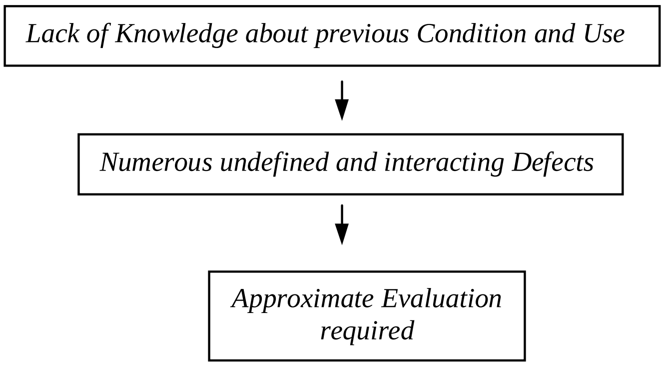

Figure 5-5: Flow-Chart for a Screening Assessment (see also Figure 4-13)

After having recorded the exact geometry including all cross-sections and dimensions in plan and elevation important bearing elements are investigated for material and structural damage using selected damage indices.

The following fault-tree aims at a *qualitative*[34] illustration of ways of, how to emphasize damage effects with a high/low correction vector K. Besides, it illustrates how to describe a strong and a weak relationship between damage cause and damage effect (Relation Matrix R).

[34] quantitative evalution is avoided since the chosen example is of theoretical nature

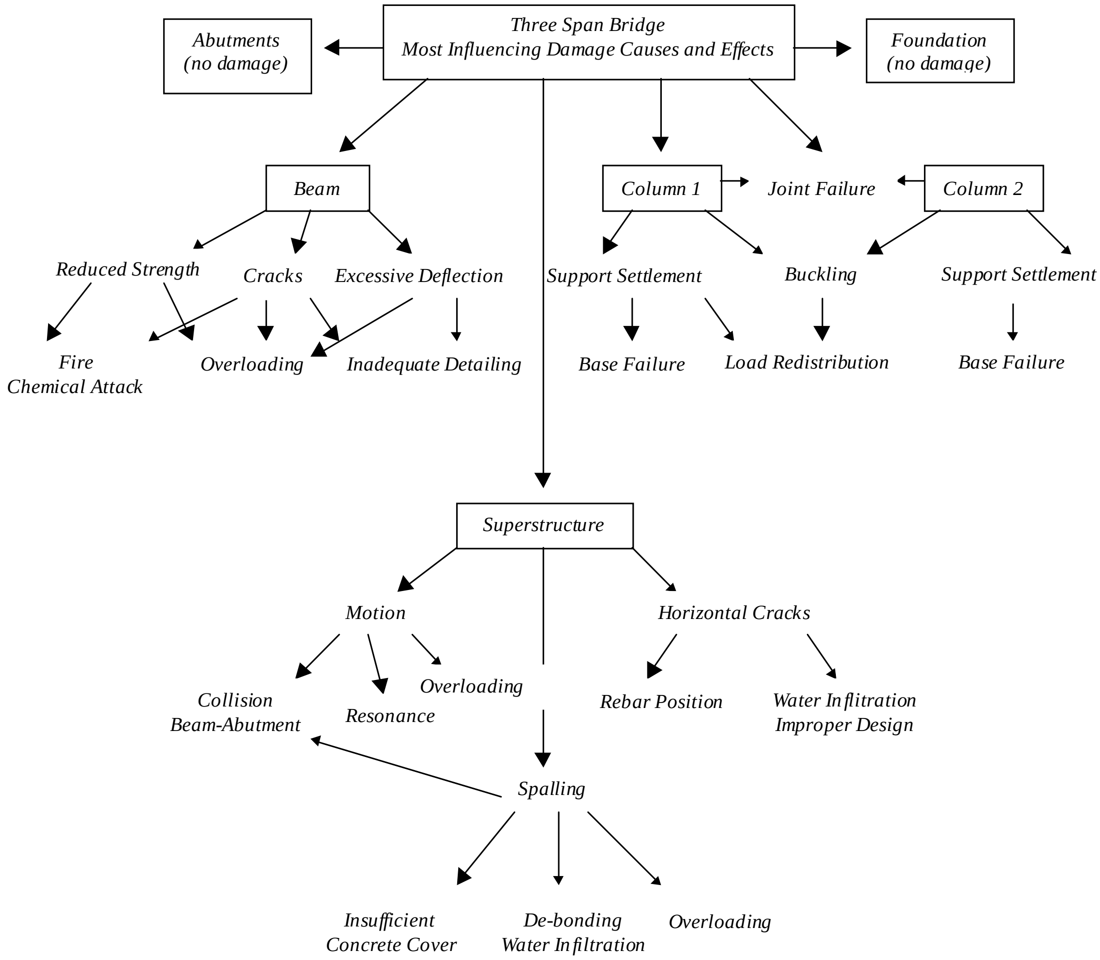

Figure 5-6: Simplified Fault-Tree for the Example Three-Span Bridge

Although having applied a fault-tree in accordance to Figure 5-6 to systematically capture all relevant defects, the experts are not sure about dominant failure modes with respect to the superstructure and with respect to the beam. Countless damage causes and effects have been revealed, each of them being subjected to random and nonrandom uncertainties -not to speak of incompatible opinions of several experts emphasizing different aspects or characterizing different local and global damage indices. Besides, large search spaces of causes and effects can no more be captured with human eyes. This is not true for column 1 and column 2 mainly suffering from base failure and an absence of sufficient load redistribution, respectively. Nevertheless, not only the superstructure and the beam are subjected to further investigations, but also column 1 and column 2 in order to guarantee a holistc illustration of the whole evaluation methodology.

Geometry Recording
Identification of Damage Parameters and Failure Modes
(Application of a Fault-Tree)

Explicit Knowledge about the Condition of Column 1 and Column 2
Uncertainty about dominant Failure Modes of the Superstructure and the Beam

Further Investigations required

Figure 5-7: Flow-Chart for an Approximate Evaluation (see also Figure 4-16)

Further investigations are performed to locate reserves for a potential load transfer away from weak areas. Questionnaires accomplished by experts combined with specified calculations based on fuzzy-logic help identify the main fracture causes and thus the predominant failure mode. Since we are not sure about the damage extent η (equation 55), the procedure based on fuzzy-logic according to chapter 4.4 will be applied:

Whereas the Relation space R (equation 71) relates all relevant damage causes V_i to the damage effects q_i, the importance of the characteristics relative to others is described using the importance space H (equation 73). The Correction Vector $K = (k_1, k_2 ..... k_r)$ describing that an effect q_i is actually revealed shall account for the degree of agreement with respect to a given statement (very true, true, fairly true, fairly false, false, very false). The matrix R and the vector K can be established using data from visual, non-destructive or laboratory investigations they also depend on the personal opinion. The latter are fuzzified and then transformed into singletons μ_{ij} and k_i values defining the cause-effect relationship of damage and the belief that a certain damage effect actually is exhibited, respectively.

The system to be analyzed represents a three-span bridge. Instead of a box girder as is shown in Figure 5-3 the example will be calculated using data from a simplified cross-section (rectangular beam) in order to better illustrate the fuzzy-algorithm (see Figure 5-4). So, the actual system consists of two abutments (no damage assumed), the foundation (no damage assumed), the superstructure, the beam, column 1 and column 2. Note, that the relation matrix R, the importance matrix H and the correction vector K consist of singletons being established by using data from visual, non-destructive and laboratory investigations interpreted with the help of expert knowledge or engineering experience. The weighting vector W^*, the weighted Hamming distance $d_w(V_i,V_j)$ and the confirmation degree C are, however, calculated:

For the **superstructure** of the three span beam bridge the first step is to establish the Relation Space according to Table 5-1. Some probable scenarios have been specified in order to introduce sufficient variability to demonstrate the approach. Assume nine possible damage causes and eight damage effects. Four different types of cracks with respect to the direction are distinguished. The damage cause “concrete corrosion” due to physicochemical disintegration and rebar corrosion due to electrochemical deterioration will be left out in this example, since here it is assumed to be not predominant and, since there is enough available literature (note, that usually concrete and rebar deficiencies *should* be differentiated, since the condition of reinforcement steel exerts a more significant influence on structural behaviour than e.g. concrete cover). Fatigue may be considered as damage effect due to e.g. overloading or resonance. However, it might as well represent a damage cause in turn leading to e.g. an instable crack propagation. The fact that there might be two different opinions with respect to the actual situation again underlines the importance of a reliable and robust mathematical algorithm capable of explicitly considering uncertainties.

In the following, the matrix R for the relationship between damage cause V_i and damage effects q_i is established dependent on the inferences drawn from the results obtained from geometry recording and material investigation (see Table 5-1). Here the horizontal axis (consisting of the rows) represents the damage cause V_i, while the vertical axis summarizes possible effects q_i. This matrix can be used to formulate the relation matrix R according to equation 71.

	Collision Beam-Abutment	*Improper Design*	*Insufficient C-Cover*	*Resonance*	*Water Infiltration*	*Fire*	*Rebar Position*	*Debonding*	*Overloading*
Motion	*0.9*	*0.2*	*0*	*0.7*	*0*	*0.1*	*0*	*0*	*0.5*
Fatigue	*0*	*0.6*	*0*	*0.6*	*0*	*0.1*	*0.3*	*0*	*0.9*
Spalling	*0.5*	*0.1*	*0.9*	*0.2*	*0.7*	*0.5*	*0.5*	*0.6*	*0.6*
T-Crack	*0*	*0.5*	*0.2*	*0*	*0.5*	*0.9*	*0.3*	*0.3*	*0.1*
V-Crack	*0.5*	*0.6*	*0.5*	*0.5*	*0.5*	*0*	*0.4*	*0.9*	*0.8*
H-Crack	*0.2*	*0.5*	*0.2*	*0.2*	*0.5*	*0.3*	*0.8*	*0*	*0*
D-Crack	*0.2*	*0.9*	*0*	*0.1*	*0.5*	*0*	*0.2*	*0.1*	*0.8*
Deflection	*0*	*0.2*	*0*	*0.5*	*0*	*0*	*0.1*	*0*	*0.8*

Table 5-1: Relationship between Damage Cause and Effect

$$R = \begin{array}{c c} & \begin{array}{cccc} V_1 & ..... & & V_n \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ . \\ q_m \end{array} & \begin{bmatrix} \mu_{11} & . & . & \mu_{1n} \\ \mu_{21} & . & . & \mu_{2n} \\ . & . & . & . \\ \mu_{m1} & . & . & \mu_{mn} \end{bmatrix} \end{array} \qquad \text{(equation 71)}$$

$$R = \begin{array}{c c} & \begin{array}{ccccccccc} V_1 & V_2 & V_3 & V_4 & V_5 & V_6 & V_7 & V_8 & V_9 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ q_6 \\ q_7 \\ q_8 \end{array} & \begin{bmatrix} 0.9 & 0.2 & 0 & 0.7 & 0 & 0.1 & 0 & 0 & 0.5 \\ 0 & 0.6 & 0 & 0.6 & 0 & 0.1 & 0.3 & 0 & 0.9 \\ 0.5 & 0.1 & 0.9 & 0.2 & 0.7 & 0.5 & 0.5 & 0.6 & 0.6 \\ 0 & 0.5 & 0.2 & 0 & 0.5 & 0.9 & 0.3 & 0.3 & 0.1 \\ 0.5 & 0.6 & 0.5 & 0.5 & 0.5 & 0 & 0.4 & 0.9 & 0.8 \\ 0.2 & 0.5 & 0.2 & 0.2 & 0.5 & 0.3 & 0.8 & 0 & 0 \\ 0.2 & 0.9 & 0 & 0.1 & 0.5 & 0 & 0.2 & 0.1 & 0.8 \\ 0 & 0.2 & 0 & 0.5 & 0 & 0 & 0.1 & 0 & 0.8 \end{bmatrix} \end{array}$$

Matrix R can be established using data from visual, non-destructive or laboratory investigation which are interpreted with the help of expert knowledge and engineering experience. The extent to which e.g. the damage effect “Motion” can be related to the damage cause “Collision Beam-Abutment” can be described using the singleton $\mu_{11} = 0.9$, while the relationship “Motion” to “Overloading” only is assigned with $\mu_{19} = 0.5$. The damage effect “Spalling” and the damage cause “Water Infiltration” are assigned using the singleton $\mu_{35} = 0.7$ etc.

The vectors for the damage causes with respect to the superstructure of the three-span beam bridge can be described with:

$$
\begin{aligned}
V_1 &= (0.9 / 0 / 0.5 / 0 / 0.5 / 0.2 / 0.2 / 0) \\
V_2 &= (0.2 / 0.6 / 0.1 / 0.5 / 0.6 / 0.5 / 0.9 / 0.2) \\
V_3 &= (0 / 0 / 0.9 / 0.2 / 0.5 / 0.2 / 0 / 0) \\
V_4 &= (0.7 / 0.6 / 0.2 / 0 / 0.5 / 0.2 / 0.1 / 0.5) \\
V_5 &= (0 / 0 / 0.7 / 0.5 / 0.5 / 0.5 / 0.5 / 0) \\
V_6 &= (0.1 / 0.1 / 0.5 / 0.9 / 0 / 0.3 / 0 / 0) \\
V_7 &= (0 / 0.3 / 0.5 / 0.3 / 0.4 / 0.8 / 0.2 / 0.1) \\
V_8 &= (0 / 0 / 0.6 / 0.3 / 0.9 / 0 / 0.1 / 0) \\
V_9 &= (0.5 / 0.9 / 0.6 / 0.1 / 0.8 / 0 / 0.8 / 0.8)
\end{aligned}
$$

Using again results from geometry recording and material investigations the next step is to relate the damage effects q_i (weight them against each other) with respect to their importance (Table 5-2). According to the importance of one damage effect q_i relative to another q_j establish the Importance Space H.

For example is the damage effect “Motion” ten times ($h_{18} = 10$) as much important as the damage effect “Deflection”. “Spalling” and “Transversal Cracks” are considered to be eight times as much important as “Deflection” ($h_{38} = h_{48} = 8$) etc.

	Motion	*Fatigue*	*Spalling*	*T-Crack*	*V-Crack*	*H-Crack*	*D-Crack*	*Deflection*
Motion	*1*	*4*	*2*	*2*	*2*	*1*	*2*	*10*
Fatigue	*0.25*	*1*	*0.5*	*0.5*	*0.5*	*0.2*	*1*	*1*
Spalling	*0.5*	*2*	*1*	*1*	*1*	*1*	*1*	*8*
T-Crack	*0.5*	*2*	*1*	*1*	*1*	*1*	*1*	*8*
V-Crack	*0.5*	*2*	*1*	*1*	*1*	*1*	*1*	*2*
H-Crack	*1*	*5*	*1*	*1*	*1*	*1*	*1*	*8*
D-Crack	*0.5*	*1*	*1*	*1*	*1*	*1*	*1*	*2*
Deflection	*0.1*	*1*	*0.125*	*0.125*	*0.5*	*0.125*	*0.5*	*1*

Table 5-2: Relative Importance of the Damage Effects

So, with q_i to be h_{ij} times more important one may conclude (equation 73)

$$H = \begin{array}{c} \\ q_1 \\ q_2 \\ . \\ q_m \end{array} \begin{array}{c} \begin{array}{ccc} q_1 & ..... & q_m \end{array} \\ \begin{bmatrix} h_{11} & . & . & h_{1m} \\ h_{21} & . & . & h_{2m} \\ . & . & . & . \\ h_{m1} & . & . & h_{mm} \end{bmatrix} \end{array} \qquad \text{(equation 73)}$$

$$H = \begin{array}{c|cccccccc|} & q_1 & q_2 & q_3 & q_4 & q_5 & q_6 & q_7 & q_8 \\ \hline q_1 & 1 & 4 & 2 & 2 & 2 & 1 & 2 & 10 \\ q_2 & 0.25 & 1 & 0.5 & 0.5 & 0.5 & 0.2 & 1 & 1 \\ q_3 & 0.5 & 2 & 1 & 1 & 1 & 1 & 1 & 8 \\ q_4 & 0.5 & 2 & 1 & 1 & 1 & 1 & 1 & 8 \\ q_5 & 0.5 & 2 & 1 & 1 & 1 & 1 & 1 & 2 \\ q_6 & 1 & 5 & 1 & 1 & 1 & 1 & 1 & 8 \\ q_7 & 0.5 & 1 & 1 & 1 & 1 & 1 & 1 & 2 \\ q_8 & 0.1 & 1 & 0.125 & 0.125 & 0.5 & 0.125 & 0.5 & 1 \end{array}$$

The weighting vector has to be calculated as the sum of the eight rows (equation 74) before the weighted Hamming distance d_w (equation 75) is determined with $\sum_{i=1}^{m} w^*_i = 1$ and $w^*_i > 0$

$$W^* = \left(\sum_{j=1}^{m} h_{1j}, \sum_{j=1}^{m} h_{2j}, ..., \sum_{j=1}^{m} h_{mj}, \right) = \left(w^*_1, w^*_2, ...w^*_m \right) \qquad \text{(equation 74)}$$

$$W^* = (24 / 5 / 15.5 / 15.5 / 9.5 / 19 / 8.5 / 3.5)$$
$$\sum 100.5$$
$$normalized\ W^* = (0.24 / 0.05 / 0.15 / 0.15 / 0.1 / 0.19 / 0.09 / 0.03)$$

The normalized weighting vector W^* may be obtained by dividing the values of the weighting vector W^* with the sum of its components: $\left(\frac{w^*_i}{\sum \left(w^*_1 + w^*_2 + ... + w^*_8 \right)} \right)$. The cause vectors V_i and V_j of each fuzzy pair pattern are now compared using the weighted Hamming distance d_w to identify dominant causes

$$d_w(V_i, V_j) = \sum_{k=1}^{m} w^*_k \left[\mu_{ki} - \mu_{kj} \right] \qquad \text{(equation 75)}$$

If $d_w(V_i, V_j) > 0$ the damage cause V_i is selected, while for $d_w(V_i, V_j) < 0$

V_j is relevant. For $d_w(V_i, V_j) = 0$ both V_i and V_j are selected. Each process step screens out one pattern identifying the other as the cause.

$$d_w(V_1, V_2) = w_1(\mu_{11} - \mu_{12}) + w_2(\mu_{21} - \mu_{22}) + w_3(\mu_{31} - \mu_{32}) + w_4(\mu_{41} - \mu_{42}) + ...w_8(\mu_{81} - \mu_{82})$$

$$\begin{aligned} d_w(V_1 / V_2) &= 0.24(0.9-0.2) + 0.05(0-0.6) + 0.15(0.5-0.1) + 0.15(0-0.5) + \\ &\quad 0.1(0.5-0.6) + 0.19(0.2-0.5) + 0.09(0.2-0.9) + 0.03(0-0.2) \\ &= 0.17 - 0.03 + 0.06 - 0.075 - 0.01 - 0.057 - 0.063 - 0.006 \\ &= -0.01 \end{aligned}$$

Since the result is below zero, V_2 is relevant though being very near to V_1 and so, V_1 can be cut off from the subsequent calculations. In the following calculations V_2, is compared with the vectors V_3 to V_9.

$$\begin{aligned} d_w(V_2 / V_3) &= 0.24(0.2-0) + 0.05(0.6-0) + 0.15(0.1-0.9) + 0.15(0.5-0.2) + \\ &\quad 0.1(0.6-0.5) + 0.19(0.5-0.2) + 0.09(0.9-0) + 0.03(0.2-0) \\ &= 0.048 + 0.03 - 0.12 + 0.045 + 0.01 + 0.057 + 0.081 + 0.006 \\ &= 0.157 \end{aligned}$$

$$\begin{aligned} d_w(V_2 / V_4) &= 0.24(0.2-0.7) + 0.05(0.6-0.6) + 0.15(0.1-0.2) + 0.15(0.5-0) - \\ &\quad 0.1(0.6-0.5) + 0.19(0.5-0.2) + 0.09(0.9-0.1) + 0.03(0.2-0.5) \\ &= -0.12 + 0 + 0.015 + 0.075 + 0.01 + 0.057 + 0.072 - 0.009 \\ &= 0.1 \end{aligned}$$

$$\begin{aligned} d_w(V_2 / V_5) &= 0.24(0.2-0) + 0.05(0.6-0) + 0.15(0.1-0.7) + 0.15(0.5-0.5) + \\ &\quad 0.1(0.6-0.5) + 0.19(0.5-0.5) + 0.09(0.9-0.5) + 0.03(0.2-0) \\ &= 0.048 - 0.03 - 0.09 + 0 + 0.01 + 0 + 0.036 + 0.06 \\ &= 0.274 \end{aligned}$$

$$\begin{aligned} d_w(V_2 / V_6) &= 0.24(0.2-0.1) + 0.05(0.6-0.1) + 0.15(0.1-0.5) + 0.15(0.5-0.9) + \\ &\quad 0.1(0.6-0) + 0.19(0.5-0.3) + 0.09(0.9-0) + 0.03(0.2-0) \\ &= 0.024 + 0.025 - 0.06 - 0.06 + 0.06 + 0.038 + 0.081 + 0.006 \\ &= 0.114 \end{aligned}$$

$$\begin{aligned} d_w(V_2/V_7) &= 0.24(0.2-0)+0.05(0.6-0.3)+0.15(0.1-0.5)+0.15(0.5-0.3)+ \\ &\quad 0.1(0.6-0.4)+0.19(0.5-0.8)+0.09(0.9-0.2)+0.03(0.2-0.1) \\ &= 0.048+0.015-0.06+0.03+0.02-0.057+0.063+0.003 \\ &= 0.062 \end{aligned}$$

$$\begin{aligned} d_w(V_2/V_8) &= 0.24(0.2-0)+0.05(0.6-0)+0.15(0.1-0.6)+0.15(0.5-0.3)+ \\ &\quad 0.1(0.6-0.9)+0.19(0.5-0)+0.09(0.9-0.1)+0.03(0.2-0) \\ &= 0.048+0-0.03-0.075+0.03-0.03+0.095+0.07 \\ &= 0.108 \end{aligned}$$

$$\begin{aligned} d_w(V_2/V_9) &= 0.24(0.2-0.5)+0.05(0.6-0.9)+0.15(0.1-0.6)+0.15(0.5-0.1)+ \\ &\quad 0.1(0.6-0.8)+0.19(0.5-0)+0.09(0.9-0.8)+0.03(0.2-0.8) \\ &= -0.072-0.015-0.075+0.06-0.02+0.095+0.009-0.018 \\ &= -0.036 \end{aligned}$$

Since the resulting value is below zero $V_j = V_9$ is relevant rather than $V_i = V_2$. Consequently, in the following calculations V_9 has to be compared only with the vectors V_3 to V_8:

$$\begin{aligned} d_w(V_9/V_3) &= 0.24(0.5-0)+0.05(0.9-0)+0.15(0.6-0.9)+0.15(0.1-0.2)+ \\ &\quad 0.1(0.8-0.5)+0.19(0-0.2)+0.09(0.8-0)+0.03(0.8-0) \\ &= 0.12+0.045-0.045-0.015+0.03-0.038+0.072+0.024 \\ &= 0.193 \end{aligned}$$

$$\begin{aligned} d_w(V_9/V_4) &= 0.24(0.5-0.7)+0.05(0.9-0.6)+0.15(0.6-0.2)+0.15(0.1-0)+ \\ &\quad 0.1(0.8-0.5)+0.19(0-0.2)+0.09(0.8-0.1)+0.03(0.8-0.5) \\ &= -0.048+0.015+0.06+0.015+0.03-0.038+0.063+0.009 \\ &= 0.106 \end{aligned}$$

$$
\begin{aligned}
d_w(V_9/V_5) = {} & 0.24(0.5-0)+0.05(0.9-0)+0.15(0.6-0.7)+0.15(0.1-0.5)+ \\
& 0.1(0.8-0.5)+0.19(0-0.5)+0.09(0.8-0.5)+0.03(0.8-0) \\
& = 0.12+0.045-0.015-0.06+0.03-0.095+0.027+0.024 \\
& = 0.076
\end{aligned}
$$

$$
\begin{aligned}
d_w(V_9/V_6) = {} & 0.24(0.5-0.1)+0.05(0.9-0.1)+0.15(0.6-0.5)+0.15(0.1-0.9)+ \\
& 0.1(0.8-0)+0.19(0-0.3)+0.09(0.8-0)+0.03(0.8-0) \\
& = 0.096+0.04+0.015-0.12+0.08-0.038+0.072+0.024 \\
& = 0.169
\end{aligned}
$$

$$
\begin{aligned}
d_w(V_9/V_7) = {} & 0.24(0.5-0)+0.05(0.9-0.3)+0.15(0.6-0.5)+0.15(0.1-0.3)+ \\
& 0.1(0.8-0.4)+0.19(0-0.8)+0.09(0.8-0.2)+0.03(0.8-0.1) \\
& = 0.12+0.03+0.015-0.03+0.04-0.152+0.054+0.021 \\
& = 0.098
\end{aligned}
$$

$$
\begin{aligned}
d_w(V_9/V_8) = {} & 0.24(0.5-0)+0.05(0.9-0)+0.15(0.6-0.6)+0.15(0.1-0.3)+ \\
& 0.1(0.8-0.9)+0.19(0-0)+0.09(0.8-0.1)+0.03(0.8-0) \\
& = 0.12+0.045+0-0.03-0.01+0+0.072+0.024 \\
& = 0.221
\end{aligned}
$$

The calculations reveal that the damage of the superstructure is mainly due to V_9 (overloading) and not due to V_8, since $d_w \geq 0$. However, damage may be attributed also to V_2 (improper design in general), since with its weighted Hamming distance $d_w(V_2/V_9) = -0.036$ it is very close to zero and to V_1 (collision between beam and abutment), whose weighted Hamming distance $d_w(V_1/V_2) = -0.01$ shows that it is close to V_2.

V_9 is considered to be mainly relevant, however directly followed by V_2 and then by V_1. The chronological order of the remaining damage causes V_5 to V_8 may be deduced from the previously calculated d_w- values. Since $d_w(V_9/V_5) = 0.076$ is below $d_w(V_9/V_7) = 0.098$, the damage cause V_5 comes before V_7. The latter comes before V_4, since

$d_w(V_9/V_7) = 0.098 < d_w(V_9/V_4) = 0.106$.

$$V_9 \rightarrow V_2 \rightarrow V_1 \rightarrow V_5 \rightarrow V_7 \rightarrow V_4 \rightarrow V_3 \rightarrow V_8$$

Again, the chronological order of the damage causes can be deduced from the values d_w. They help select the weights w_i with which the damage indices D_i have to be multiplied in order to obtain the damage extent $\eta = \frac{(D_i / \gamma)^2 \times w_i}{\sum D_i^2}$.

The degree of agreement for the statement that a certain damage effect q_i actually is exhibited now is described with the help of the correction vector K. So does $K = (0.9 / 0.4 / 0.6 / 0.2 / 0.2 / 0.4 / 0.2 / 0)$ represent a level of confidence / agreement that the damage effects "Motion", "Fatigue", "Spalling" etc. are probable to an extent of *0.9, 0.4, 0.6 etc.*, respectively.

Dependent on the normalized weighting vector W^*, the correction vector K and the cause vector V_9 the extent to which damage cause V_9 is relevant can be calculated with the confirmation degree for cause no 9:

$$C_i = \begin{pmatrix} W^*_1 \times K_1 \\ W^*_2 \times K_2 \\ \vdots \quad \vdots \\ W^*_n \times K_n \end{pmatrix} \times \begin{pmatrix} V_{i1} \\ V_{i2} \\ \vdots \\ V_{in} \end{pmatrix} \qquad \text{(equation 76)}$$

$$\begin{aligned} C_{V9} &= \sum (W^* \times K) V_9 \\ &= 0.24 \times 0.9 \times 0.5 + 0.05 \times 0.4 \times 0.9 + 0.15 \times 0.6 \times 0.6 + 0.15 \times 0.2 \times 0.1 + \\ &\quad 0.1 \times 0.2 \times 0.8 + 0.19 \times 0.4 \times 0 + 0.09 \times 0.2 \times 0.8 + 0.03 \times 0 \times 0.8 \\ &= 0.108 + 0.018 + 0.054 + 0.003 + 0.016 + 0.014 \\ &= 0.213 \end{aligned}$$

The relevant damage cause V_9 "overloading" for the superstructure is confirmed to a degree of 21.3 %.

Beam

In the same way as for the superstructure, also for the beam the values shown in Table 5-3 and representing the relationship between damage cause V_i and effect q_i are established dependent on inferences drawn from previous geometry recording and material investigation. Again, the horizontal axis represents the damage cause V_i, while the vertical axis summarizes possible effects q_i. This matrix can be used to formulate the relation matrix according to equation 71.

The damage cause V_4 "inadequate detailing" includes e.g. inadequate anchorage of longitudinal or transfer rebars leading to a bond slipping failure and finally to excessive cracking over the supports. The values of the relation space R are defined by the expert performing the damage assessment. He votes that for both, the damage effects q_1 "Reduced Strength" and q_2 "Section Loss" one may consider responsible the damage causes V_1 "Chemical Attack" and V_2 "Fire". This attitude can be confirmed by engineering experience with respect to reinforced concrete structures. "Reduced "Strength" is much more related to "Chemical Attack" and "Fire" than e.g. with "Overloading" itself mainly causing "cracks" and "Excessive Deflection".

	Chemical Attack	*Fire*	*Overloading*	*Inadequate Detailing*
Reduced Strength	*0.9*	*0.9*	*0.1*	*0.3*
Section Loss	*0.9*	*0.9*	*0.3*	*0.2*
Cracks	*0.6*	*0.7*	*0.9*	*0.9*
Excessive Deflection	*0.1*	*0.4*	*0.9*	*0.5*

Table 5-3: Relationship between Damage Cause and Effect

$$Relation\ Space\ R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \\ q_4 \end{array} \begin{array}{c} \begin{array}{cccc} V_1 & V_2 & V_3 & V_4 \end{array} \\ \begin{bmatrix} \mu_{11} & \mu_{12} & \mu_{13} & \mu_{14} \\ \mu_{21} & \mu_{22} & \mu_{23} & \mu_{24} \\ \mu_{31} & \mu_{32} & \mu_{33} & \mu_{34} \\ \mu_{41} & \mu_{42} & \mu_{43} & \mu_{44} \end{bmatrix} \end{array} \qquad \text{(equation 71)}$$

$$Relation\ Space\ R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \\ q_4 \end{array} \begin{array}{c} \begin{array}{cccc} V_1 & V_2 & V_3 & V_4 \end{array} \\ \begin{bmatrix} 0.9 & 0.9 & 0.1 & 0.3 \\ 0.9 & 0.9 & 0.3 & 0.2 \\ 0.6 & 0.7 & 0.9 & 0.9 \\ 0.1 & 0.4 & 0.9 & 0.5 \end{bmatrix} \end{array}$$

The extent to which e.g. the damage effect "Reduced Strength" and "Section Loss" can be related to the damage cause "Chemical Attack" using the singleton $\mu_{11} = \mu_{21} = 0.9$. The same applies to the relationship "Excessive Deflection" with respect to "Overloading" ($\mu_{33} = \mu_{43} = 0.9$).

The possible causes for failure can be represented with the help of the vectors

$$V_1 = (0.9 / 0.9 / 0.6 / 0.1)$$
$$V_2 = (0.9 / 0.9 / 0.7 / 0.4)$$
$$V_3 = (0.1 / 0.3 / 0.9 / 0.9)$$
$$V_4 = (0.3 / 0.2 / 0.9 / 0.5)$$

Using again results from geometry recording and material investigations the next step is to relate the damage effects q_i (weight them against each other) with respect to their importance (Table 5-4). According to the importance of one damage effect relative to another one may establish the Importance Space H:

	Reduced Strength	*Section Loss*	*Cracks*	*Excessive Deflection*
Reduced Strength	*1*	*2*	*0.1*	*0.1*
Section Loss	*0.5*	*1*	*0.25*	*0.25*
Cracks	*10*	*4*	*1*	*1*
Excessive Deflection	*10*	*4*	*1*	*1*

Table 5-4: Relative Importance of the Damage Effects

The damage effects "Cracks" and "Excessive Deflection" are considered to be ten times as much important ($h_{31} = h_{41} = 10$) as the damage effect "Reduced Strength" and four times ($h_{32} = h_{42} = 4$) as much important as "Section Loss". So, with q_i to be h_{ij} times more important than q_j one may establish H with:

$$Importance\ Space\ H = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \\ q_4 \end{array} \begin{array}{c} \begin{array}{cccc} q_1 & q_2 & q_3 & q_4 \end{array} \\ \begin{bmatrix} h_{11} & h_{12} & h_{13} & h_{14} \\ h_{21} & h_{22} & h_{23} & h_{24} \\ h_{31} & h_{32} & h_{33} & h_{34} \\ h_{41} & h_{42} & h_{43} & h_{44} \end{bmatrix} \end{array} \qquad \text{(equation 73)}$$

one may conclude

$$Importance\ \ Space\ H = \begin{array}{c|cccc} & q_1 & q_2 & q_3 & q_4 \\ \hline q_1 & 1 & 2 & .1 & .1 \\ q_2 & .5 & 1 & .25 & .25 \\ q_3 & 10 & 4 & 1 & 1 \\ q_4 & 10 & 4 & 1 & 1 \end{array}$$

The weighted Hamming distance d_w depends on the weighting vector

$$\begin{aligned} &W^* = (3.2 / 2 / 16 / 16) \quad \sum 37.2 \\ &normalized \quad W^* = (0.09 / 0.05 / 0.43 / 0.43) \end{aligned} \qquad \text{(equation 74)}$$

In the same way as in case of the superstructure, the cause vectors V_i and V_j of each fuzzy pair pattern are now compared using the weighted Hamming distance d_w (equation 75) to identify the dominant cause.

$$d_w(V_i, V_j) = \sum_{k=1}^{m} w^*_k \left[\mu_{ki} - \mu_{kj}\right] \qquad \text{(equation 75)}$$

If $d_w(V_i, V_j) > 0$ the damage cause V_i is selected, while for $d_w(V_i, V_j) < 0$

V_j is relevant. For $d_w(V_i, V_j) = 0$ both V_i and V_j are selected. Each process step screens out one pattern identifying the other as the cause.

$$d_w(V_1 / V_2) = 0.09 \times (0.9 - 0.9) + 0.05 \times (0.9 - 0.9) + 0.43 \times (0.6 - 0.7) + 0.43 \times (0.1 - 0.4) = -0.172$$

Since d_w is below zero the damage cause, V_2 is relevant and now has to be compared with the damage causes V_3 and V_4:

$$d_w(V_2 / V_3) = 0.09 \times (0.9 - 0.1) + 0.05 \times (0.9 - 0.3) + 0.43(0.7 - 0.9) + 0.43(0.4 - 0.9) = -0.20$$

V_3 is relevant and now has to be compared with V_4:

$$d_w(V_3 / V_4) = 0.09 \times (0.1 - 0.3) + 0.05 \times (0.3 - 0.2) + 0.43(0.9 - 0.9) + 0.43(0.9 - 0.5) = 0.16$$

Also in the last comparison the damage cause V_3 (overloading) is relevant. The chronological order of the remaining damage causes may be deduced from the previously calculated d_w - values:

$$V_3 \rightarrow V_4 \rightarrow V_2 \rightarrow V_1$$

The damage cause V_4 directly follows V_3 (before V_2), since the weighted Hamming distance $d_w(V_3 / V_4 = 0.16)$ is nearer to zero than $d_w(V_2 / V_3) = -0.2$. The correction vector $K = (0.4 / 0.2 / 0.9 / 0.9)$ represents the degree of agreement for the statement that the damage effects “ Reduced Strength”, “Section Loss”, “Cracks” and “Excessive Deflection” actually are exhibited (corresponds to *0.4*, *0.2*, *0.9* and *0.9*, respectively). So, dependent on K, W^* and the damage cause $V_3 = (0.1 / 0.3 / 0.9 / 0.9)$ the confirmation degree for cause no. 3 results in

$$C_i = \begin{pmatrix} W^*_1 \times K_1 \\ W^*_2 \times K_2 \\ \vdots \quad \vdots \\ W^*_n \times K_n \end{pmatrix} \times \begin{pmatrix} V_{i1} \\ V_{i2} \\ \vdots \\ V_{in} \end{pmatrix} \qquad \text{(equation 76)}$$

$$\begin{aligned} C_{V3} &= \sum_{j=1}^{4} (W^* \times K) \times V_3 \\ &= 0.09 \times 0.4 \times 0.1 + 0.05 \times 0.2 \times 0.3 + 0.43 \times 0.9 \times 0.9 + 0.43 \times 0.9 \times 0.9 \\ &= 0.70 \end{aligned}$$

The relevant damage cause V_3 “overloading” for the beam is confirmed to a degree of 70%.

<u>Column 1</u>

In the same way as for the superstructure and for the beam, also for column 1 the values shown in Table 5-5 representing the relationship between damage cause V_i and effect q_i is established dependent on inferences drawn from previous geometry recording and material investigation. Again, the horizontal axis (consisting of the rows) represents the damage cause V_i, while the vertical axis summarizes possible effects q_i. This matrix can be used to formulate the relation matrix R according to equation 71.

	Load Redistribution	*Inadequate Detailing*	*Base Failure*
Buckling	*0.9*	*0.3*	*0.1*
Joint Failure	*0.9*	*0.9*	*0.1*
Support Settlement	*0.1*	*0.1*	*0.9*

Table 5-5: Relationship between Damage Cause and Damage Effect

$$Relation\,Space\,R = \begin{array}{c c} & \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} \mu_{11} & \mu_{12} & \mu_{13} \\ \mu_{21} & \mu_{22} & \mu_{23} \\ \mu_{31} & \mu_{32} & \mu_{33} \end{bmatrix} \end{array} \qquad \text{(equation 71)}$$

$$Relation\,Space\,R = \begin{array}{c c} & \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 0.9 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.1 & 0.1 & 0.9 \end{bmatrix} \end{array}$$

The extent to which e.g. the damage effects “Buckling” and “Joint Failure” can be related to the damage cause “Load Redistribution” using the singleton $\mu_{11} = \mu_{21} = 0.9$, while the relationship “Support Settlement” to “ Load Redistribution” only is assigned with $\mu_{31} = 0.1$.

The possible causes for failure can be represented with the vectors

$$V_1 = (0.9 / 0.9 / 0.1)$$
$$V_2 = (0.3 / 0.9 / 0.1)$$
$$V_3 = (0.1 / 0.1 / 0.9)$$

Using again results from geometry recording and material investigations now relate the damage effects q_i (weight them against each other) with respect to their importance (Table 5-6). According to the importance of a damage effect q_i relative to another one may establish the Importance Space H:

	Buckling	*Joint Failure*	*Support Settlement*
Buckling	*1*	*0.25*	*0.1*
Joint Failure	*4*	*1*	*0.25*
Support Settlement	*10*	*4*	*1*

Table 5-6: Relative Importance of the Damage Effects

The damage effect "Joint Failure" is considered to be four times ($h_{21} = 4$) and "Support Settlement" ten times ($h_{31} = 10$) as much important as the damage effect "Buckling". So, with q_i to be h_{ij} times more important than q_j one may establish the importance space H with

$$Importance\,Space\,H = \begin{array}{c|ccc} & q_1 & q_2 & q_3 \\ \hline q_1 & h_{11} & h_{12} & h_{13} \\ q_2 & h_{21} & h_{22} & h_{23} \\ q_3 & h_{31} & h_{32} & h_{33} \end{array} \qquad \text{(equation 73)}$$

$$Importance\,Space\,H = \begin{array}{c|ccc} & q_1 & q_2 & q_3 \\ \hline q_1 & 1 & .25 & .1 \\ q_2 & 4 & 1 & .25 \\ q_3 & 10 & 4 & 1 \end{array}$$

The weighting vector necessary to calculate the weighted Hamming distance is

$$\begin{aligned} &W^* = (1.35 / 5.25 / 15) \quad \sum 21.6 \\ &normalized \quad W^* = (0.063 / 0.24 / 0.69) \end{aligned} \qquad \text{(equation 74)}$$

In the same way as in case of the superstructure and for the beam, the cause vectors V_i and V_j of each fuzzy pair pattern are now compared using the weighted Hamming distance d_w to identify the dominant cause.

$$d_w(V_i, V_j) = \sum_{k=1}^{m} w^*_k \left[\mu_{ki} - \mu_{kj}\right] \qquad \text{(equation 75)}$$

If $d_w(V_i, V_j) > 0$ the damage cause V_i is selected, while for $d_w(V_i, V_j) < 0$

V_j is relevant. For $d_w\left(V_i, V_j\right) = 0$ both V_i and V_j are selected. Each process step screens out one pattern identifying the other as the cause:

$$d_w(V_1 / V_2) = 0.063 \times (0.9 - 0.3) + 0.24 \times (0.9 - 0.9) + 0.69(0.1 - 0.1) = 0.034$$

Since $d_w\left(V_i, V_j\right) > 0$ one may assume that V_1 is relevant though (near to V_2)

$$d_w(V_1 / V_3) = 0.063 \times (0.9 - 0.1) + 0.24 \times (0.9 - 0.1) + 0.69(0.1 - 0.9) = -0.31$$

A comparison between V_i and V_j of each fuzzy pattern shows that V_3 (base failure) is relevant. The chronological order of the damage causes is:

$$V_3 \rightarrow V_1 \rightarrow V_2$$

With $d_w(V_1 / V_2) = 0.034$ (very near to zero) the damage causes V_1 and V_2 are very close to each other. The Confirmation degree C_i describes the extent to which V_3 is relevant.

With the correction vector $K = (0.1 / 0.3 / 0.8)$ describing the belief that the damage effects " Buckling", "Joint Failure" and "Support Settlement" actually are exhibited (corresponds to *0.1*, *0.3* and *0.8*, respectively), the normalized weighting vector $W^* = (0.063 / 0.24 / 0.69)$ and the damage cause $V_3 = (0.1 / 0.1 / 0.9)$ the confirmation degree (equation 76) for cause 3 results in

$$C_i = \begin{pmatrix} W^*_1 \times K_1 \\ W^*_2 \times K_2 \\ \vdots \quad \vdots \\ W^*_n \times K_n \end{pmatrix} \times \begin{pmatrix} V_{i1} \\ V_{i2} \\ \vdots \\ V_{in} \end{pmatrix}$$

$$\begin{aligned} C_{V3} &= \sum_{j=1}^{3} (W^* \times K) V_3 \\ &= (0.063 \times 0.1) 0.1 + (0.24 \times 0.3) 0.1 + (0.69 \times 0.8) 0.9 \\ &= 0.51 \end{aligned}$$

The relevant damage cause V_3 "base failure" for column 1 is confirmed to a degree of 51 %.

Column 2

In the same way as for the superstructure, the beam and column 1, also for column 2 the values shown in Table 5-7 representing the relationship between damage cause V_i and effect q_i is established dependent on inferences drawn from previous geometry recording and material investigation (equivalent to column 1). Again, the horizontal axis (consisting of the rows) represents the damage cause V_i, while the vertical axis summarizes possible effects q_i. This matrix can be used to formulate the relation matrix R according to equation 71:

	Load Redistribution	*Inadequate Detailing*	*Base Failure*
Buckling	*0.9*	*0.3*	*0.1*
Joint Failure	*0.9*	*0.9*	*0.1*
Support Settlement	*0.1*	*0.1*	*0.9*

Table 5-7: Relationship between Damage Cause and Effect

$$Relation\ Space\ R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \end{array} \begin{array}{c} \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{bmatrix} \mu_{11} & \mu_{12} & \mu_{13} \\ \mu_{21} & \mu_{22} & \mu_{23} \\ \mu_{31} & \mu_{32} & \mu_{33} \end{bmatrix} \end{array} \qquad \text{(equation 71)}$$

$$Relation\ Space\ R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \end{array} \begin{array}{c} \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{bmatrix} 0.9 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.1 & 0.1 & 0.9 \end{bmatrix} \end{array}$$

The possible causes for failure are represented with the vectors

$$V_1 = (0.9/0.9/0.1)$$
$$V_2 = (0.3/0.9/0.1)$$
$$V_3 = (0.1/0.1/0.9)$$

Using again results from geometry recording and material investigations the next step is to relate the damage effects q_i (weight them against each other) with respect to their importance (Table 5-8). According to the importance of one damage effect relative to another one may establish the Importance Space H:

With q_i to be h_{ij} times more important than q_j one may conclude

	Buckling	*Joint Failure*	*Support Settlement*
Buckling	*1*	*4*	*10*
Joint Failure	*0.25*	*1*	*4*
Support Settlement	*0.1*	*0.25*	*1*

Table 5-8: Relative Importance of the Damage Effects

Whereas the Relation Matrix R for column 2 is equivalent to column 1, this fact cannot be assumed for the Importance matrix H. For column 2 the damage effect “Buckling” can be considered to be four times ($h_{23} = 4$) as much important as the damage effect “Joint Failure” and ten times ($h_{13} = 10$) as much important as the damage effect “Support Settlement”.

$$Importance\,Space\,H = \begin{array}{c c} & \begin{array}{ccc} q_1 & q_2 & q_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} h_{11} & h_{12} & h_{13} \\ h_{21} & h_{22} & h_{23} \\ h_{31} & h_{32} & h_{33} \end{bmatrix} \end{array} \qquad \text{(equation 73)}$$

$$H = \begin{array}{c c} & \begin{array}{ccc} q_1 & q_2 & q_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 1 & 4 & 10 \\ .25 & 1 & 4 \\ .1 & .25 & 1 \end{bmatrix} \end{array}$$

The normalized weighting vector W^* is then calculated dependent on the importance values h_{ij} of the Importance Space H (calculate the sum of the rows)

$$\sum_{i=1}^{m} w^*_i = 1 \text{ and } w^*_1 > 0 \qquad W^* = \left(\sum_{j=1}^{m} h_{1j}, \sum_{j=1}^{m} h_{2j}, \ldots, \sum_{j=1}^{m} h_{mj}, \right) = \left(w^*_1, w^*_2, \ldots w^*_m \right)$$

$$\begin{aligned} & W^* = (15 / 5.25 / 1.35) \quad \sum 21.6 \\ & normalized \quad W^* = (0.69 / 0.24 / 0.063) \end{aligned} \qquad \text{(equation 74)}$$

In the same way as in case of the superstructure, the beam and column 1, the cause vectors V_i and V_j of each fuzzy pair pattern are now compared using the weighted Hamming distance d_w to identify the dominant cause

$$d_w(V_i,V_j)=\sum_{k=1}^{m} w^*{}_k\left[\mu_{ki}-\mu_{kj}\right].\qquad\text{(equation 75)}$$

If $d_w(V_i,V_j)>0$ the damage cause V_i is selected, while for $d_w(V_i,V_j)<0$

V_j is relevant. For $d_w(V_i,V_j)=0$ both V_i and V_j are selected. Each process step screens out one pattern identifying the other as the cause:

$$d_w(V_i,V_j)=\sum_{k}^{s} w^*{}_k\left[\mu_{V_i}(q_k)-\mu_{V_j}(q_k)\right].$$

$$d_w(V_1/V_2)=0.69\times(0.9-0.3)+0.24\times(0.9-0.9)+0.063(0.1-0.1)=0.414$$

Each process step screens out one pattern identifying the other as the cause. Since the result is above zero V_1 is relevant

$$d_w(V_1/V_3)=0.69\times(0.9-0.1)+0.24\times(0.9-0.1)+0.063(0.1-0.9)=0.694$$

V_1 is still relevant. The chronological order of the damage causes is

$$V_1\rightarrow V_2\rightarrow V_3$$

The damage cause V_2 comes before V_3, since

$d_w(V_1/V_2)=0.414\ <\ d_w(V_1/V_3)=0.694$.

With the normalized $W^*=(0.69/0.24/0.063)$, $V_1=(0.9/0.9/0.1)$ and $K=(0.9/0.4/0.1)$ representing a correction factor for the belief that the damage effects “Buckling”, “Joint Failure” and “Support Settlement” actually are exhibited (corresponds to *0.9, 0.4* and *0.1*, respectively), the Confirmation degree C_{V_1} for cause no. 1 results in

$$C_i=\begin{pmatrix} W^*{}_1\times K_1\\ W^*{}_2\times K_2\\ \vdots \quad \vdots\\ W^*{}_n\times K_n\end{pmatrix}\times\begin{pmatrix} V_{i1}\\ V_{i2}\\ \vdots\\ V_{in}\end{pmatrix}\qquad\text{(equation 76)}$$

$$
\begin{aligned}
C_{V1} &= \sum_{j=1}^{3} \left(W^{*} \times K\right) V_{1} \\
&= (0.69 \times 0.9)0.9 + (0.24 \times 0.4)0.9 + (0.063 \times 0.1)0.1 \\
&= 0.65
\end{aligned}
$$

The relevant damage cause V_1 “buckling“ for column 2 is confirmed to a degree of 65 %.

There is always some fuzziness in human evaluations which has to be explicitly considered. First, the results of geometry recording and material investigation can differ. Second, due to a different background or interpretation of ambiguous linguistic phrases experts express differences in opinion. The differences between two attitudes with respect to damage causes and/or damage effects (low, medium, rather high, high) may be large from the qualitative point of view, but the actual differences in quantitative measures small (e.g. few fractures of a mm in deformation). Also, even by a careful choice fuzzy-values the meaning can be unduly changed. Some experts are hesitant or confused in their response because specific information on structural parameters or serviceability was not given to them. Having only a photograph they rely on their own intuitive knowledge. So, before continuing with the calculations it is recommended to evaluate the robustness of the applied fuzzy algorithm and to check if the obtained results are correct. This can be done with the help of parameter studies which inform about the effect different parameters exert on the calculated results.

In this validation process each parameter is varied one at a time, reversing each change before proceeding to the next parameter. It is important not to accumulate changes. Parameters with cross-sensitivities accumulating effects have to be neglected. Besides, the change should consider only one expert opinion such as e.g. a slightly different view of the relationship between e.g. load redistribution and buckling, joint failure or support settlement. He may assign a different importance space H and/or another correction vector K. If the algorithm is robust its results then should not change to a high extent, provided that just a few values are varied only in one direction, and only to a small extent.

For the superstructure the two experts are assumed to have come to approximately the same values. Therefore the reliability of the fuzzy algorithm is validated only for the beam, column 1 and column 2.

Beam

The second expert thinks that section loss and reduced strength are produced by chemical attack to a lower degree (the cause vector changes with $V_1 = (0.9/0.9/0.6/0.1) \rightarrow (0.8/0.8/0.6/0.1)$), but also both more important and more probable than cracks or deflection. Besides, the second expert wants to additionally consider the damage effect “bond loss or debonding” with the same importance and probability as "reduced strength" and "section loss", however with a much lower importance and probability than "crack" and "deflection" ($K = (0.4/0.2/0.9/0.9) \to (0.5/0.3/0.9/0.9/0.3/0.3)$.

Expert 2 adds two damage effects, namely bond loss and reduced ductility. Besides, chemical attack is supposed to be less related to reduced strength and section loss. Table 5-9 shows relevant damage causes and effects:

	Chemical Attack	*Fire*	*Overloading*	*Inadequate Detailing*
Reduced Strength	*0.8*	*0.9*	*0.1*	*0.3*
Section Loss	*0.8*	*0.9*	*0.3*	*0.2*
Cracks	*0.6*	*0.7*	*0.9*	*0.9*
Excessive Deflection	*0.1*	*0.4*	*0.9*	*0.5*
Bond Loss	*0.1*	*0.3*	*0.2*	*0.9*
Reduced Ductility	*0*	*0.3*	*0.2*	*0.9*

Table 5-9: Relationship between Damage Cause and Effect

So, the Relation Space changes with (equation 71)

$$R = \begin{array}{c} q_1 \\ q_2 \\ q_3 \\ q_4 \end{array} \begin{bmatrix} 0.9 & 0.9 & 0.1 & 0.3 \\ 0.9 & 0.9 & 0.3 & 0.2 \\ 0.6 & 0.7 & 0.9 & 0.9 \\ 0.1 & 0.4 & 0.9 & 0.5 \end{bmatrix} \to \begin{array}{cc} & \begin{array}{cccc} V_1 & V_2 & V_3 & V_4 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ q_6 \end{array} & \begin{bmatrix} 0.8 & 0.9 & 0.1 & 0.3 \\ 0.8 & 0.9 & 0.3 & 0.2 \\ 0.6 & 0.7 & 0.9 & 0.9 \\ 0.1 & 0.4 & 0.9 & 0.5 \\ 0.1 & 0.3 & 0.2 & 0.9 \\ 0 & 0.3 & 0.2 & 0.9 \end{bmatrix} \end{array}$$

The causes for failure can be represented with the help of the vectors

$$\begin{aligned} V_1 &= (0.9/0.9/0.6/0.1) \;\; to \;\; V_1 = (0.8/0.8/0.6/0.1/0.1/0) \\ V_2 &= (0.9/0.9/0.7/0.4) \;\; to \;\; V_2 = (0.9/0.9/0.7/0.4/0.3/0.3) \\ V_3 &= (0.1/0.3/0.9/0.9) \;\; to \;\; V_3 = (0.1/0.3/0.9/0.9/0.2/0.2) \\ V_4 &= (0.3/0.2/0.9/0.5) \;\; to \;\; V_4 = (0.3/0.2/0.9/0.5/0.9/0.9) \end{aligned}$$

The Importance Space (equation 73) and the Weighting Vector (equation 74) change with

$$H = \begin{array}{c} q_1 \\ q_2 \\ q_3 \\ q_4 \end{array} \begin{bmatrix} 1 & 2 & .1 & .1 \\ .5 & 1 & .25 & .25 \\ 10 & 4 & 1 & 1 \\ 10 & 4 & 1 & 1 \end{bmatrix} \rightarrow \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ q_6 \end{array} \begin{array}{c} \begin{array}{cccccc} q_1 & q_2 & q_3 & q_4 & q_5 & q_6 \end{array} \\ \begin{bmatrix} 1 & 2 & 0.25 & 0.25 & 1 & 1 \\ 0.5 & 1 & 0.5 & 0.5 & 1 & 1 \\ 4 & 2 & 1 & 1 & 5 & 5 \\ 4 & 2 & 1 & 1 & 5 & 5 \\ 1 & 1 & 0.2 & 0.2 & 1 & 1 \\ 1 & 1 & 0.2 & 0.2 & 1 & 1 \end{bmatrix} \end{array}$$

$$W^* = (5.5 / 4.5 / 18 / 18 / 4.4 / 4.4)$$
$$\sum 54.8$$
$$normalized\ \ W^* = (0.10/0.08/0.33/0.33/0.08/0.08)$$

With the correction vector $K = (0.5 / 0.3 / 0.9 / 0.9 / 0.3 / 0.3)$ one should find out first if the damage cause V_3 (overloading) still is dominant by calculating the weighted Hamming distance (equation 75)

$$\begin{aligned} d_w(V_1 / V_2) = {} & 0.10 \times (0.9 - 0.9) + 0.08 \times (0.9 - 0.9) + 0.33(0.6 - 0.7) + \\ & 0.33(0.1 - 0.4) + 0.08(0.1 - 0.3) + 0.08(0 - 0.3) \\ = {} & -0.16 \end{aligned}$$

V_2 is still relevant, even to a higher extent than originally.

$$\begin{aligned} d_w(V_1 / V_3) = {} & 0.10 \times (0.9 - 0.1) + 0.08 \times (0.9 - 0.3) + 0.33(0.6 - 0.9) + \\ & 0.33(0.1 - 0.9) + 0.08(0.1 - 0.2) + 0.08(0 - 0.2) \\ = {} & -0.26 \end{aligned}$$

V_3 (overloading) is still relevant, even to a higher extent than originally.

$$\begin{aligned} d_w(V_3 / V_4) = {} & 0.10 \times (0.1 - 0.3) + 0.08 \times (0.3 - 0.2) + 0.33(0.9 - 0.9) + \\ & 0.33(0.9 - 0.5) + 0.08(0.2 - 0.9) + 0.08(0.2 - 0.9) \\ = {} & -0.008 \end{aligned}$$

V_4 (inadequate Detailing) is still relevant, while originally V_3 (Overloading) was dominant. However, both damage causes are very near and therefore assumed to be equivalent. The confirmation degree for cause V_3 and for cause V_4 results in

$$C_{V3} = \sum_{j=1}^{3} \left(W^* \times P^*\right) V_3$$
$$C_{V4} = \sum_{j=1}^{3} \left(W^* \times P^*\right) V_4 \qquad \text{(equation 76)}$$

$$\begin{aligned} C_{V3} &= (0.10 \times 0.5)0.1 + (0.08 \times 0.3)0.3 + (0.33 \times 0.9)0.9 + \\ &\quad (0.33 \times 0.9)0.9 + (0.08 \times 0.3)0.2 + (0.08 \times 0.3)0.2 \\ &= 0.56 \end{aligned}$$

$$\begin{aligned} C_{V4} &= (0.10 \times 0.5)0.3 + (0.08 \times 0.3)0.2 + (0.33 \times 0.9)0.9 + \\ &\quad (0.33 \times 0.9)0.5 + (0.08 \times 0.3)0.9 + (0.08 \times 0.3)0.9 \\ &= 0.57 \end{aligned}$$

V_3 and V_4 have a confirmation degree of *56 %* and *57 %*, respectively. This is lower compared to the first evaluation with 70 % (overloading).

The first expert thinks that column 1 mainly is damaged in form of support settlement due to ground failure, while column 2 suffers from buckling due to load redistribution. The second expert, however, considers the structural behaviour of both columns to be a little bit more similar. He considers buckling also for column 1, and support settlement also for column 2.

Column 1

According to the second expert buckling is considered to be less related to load redistribution. Besides, he considers buckling to be more important and also more probable compared to support settlement (equation 71):

$$R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \end{array} \begin{array}{c} \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{bmatrix} 0.9 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.1 & 0.1 & 0.9 \end{bmatrix} \end{array} \qquad \rightarrow \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \end{array} \begin{array}{c} \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \begin{bmatrix} 0.7 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.3 & 0.1 & 0.9 \end{bmatrix} \end{array}$$

The cause vector V_1 changes with

$$V_1 = (0.9 / 0.9 / 0.1) \;\; to \;\; V_1 = (0.7 / 0.9 / 0.3)$$

Support settlement / Buckling are less differing in their importance and so the values change with $\begin{matrix} h_{31} = 10 \rightarrow 6.7 \\ h_{13} = 0.1 \rightarrow 0.15 \end{matrix}$ (equation 73- for column 1 support settlement is assumed to be *6.7* instead of *10* times more important than buckling).

$$H = \begin{array}{c c} & \begin{array}{ccc} q_1 & q_2 & q_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 1 & .25 & .1 \\ 4 & 1 & .25 \\ 10 & 4 & 1 \end{bmatrix} \end{array} \rightarrow \begin{array}{c c} & \begin{array}{ccc} q_1 & q_2 & q_3 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 1 & .25 & .15 \\ 4 & 1 & .25 \\ 6.7 & 4 & 1 \end{bmatrix} \end{array}$$

$$K = (0.1/0.3/0.8) \rightarrow (0.2/0.3/0.7)$$

With the correction Vector $K = (0.2/0.3/0.7)$, $V_1 = (0.7/0.9/0.3)$ and the normalized weighting vector $W^* = (0.08/0.29/0.64)$ one should find out first if the damage cause V_3 still is dominant (equation 75)

$$d_w(V_1/V_2) = 0.08 \times (0.7 - 0.3) + 0.29 \times (0.9 - 0.9) + 0.64(0.3 - 0.1) = 0.16$$

The obtained results shows that V_1 is to a higher extent relevant compared to the original evaluation.

$$d_w(V_1/V_3) = 0.08 \times (0.7 - 0.1) + 0.29 \times (0.9 - 0.1) + 0.64(0.3 - 0.9) = -0.104$$

The damage cause V_3 (base failure) is still relevant, but to a lower extent compared to the original evaluation. The new confirmation degree for column 1 with respect to damage cause no. 3 results in (equation 76).

$$\begin{aligned} C_{V3} &= \sum_{j=1}^{3} (W \times P^*) V_3 \\ &= (0.08 \times 0.2) 0.1 + (0.29 \times 0.3) 0.1 + (0.64 \times 0.7) 0.9 \\ &= 0.41 \end{aligned}$$

V_3 is relevant with a probability of only 41 % compared to the first evaluation, where it was 51 % (base failure).

Column 2

The second expert assigns buckling to be less important (h_{ij}-values) and less probable (k_i-values) than support settlement (equation 71).

$$\begin{array}{cc} & \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ R = \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 0.9 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.1 & 0.1 & 0.9 \end{bmatrix} \end{array} \quad \begin{array}{cc} & \begin{array}{ccc} V_1 & V_2 & V_3 \end{array} \\ \rightarrow \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 0.7 & 0.3 & 0.1 \\ 0.9 & 0.9 & 0.1 \\ 0.3 & 0.1 & 0.9 \end{bmatrix} \end{array}$$

The cause vector V_1 changes with

$$V_1 = (0.9/0.9/0.1) \ \ to \ \ V_1 = (0.7/0.9/0.3)$$

Their importance and therefore the values change with $h_{31} = 0.1 \rightarrow 0.15$ and $h_{13} = 10 \rightarrow 6.7$ (equation 73 -buckling is assumed to be 6.7 instead of 10 times as much important as Support Settlement).

$$\begin{array}{cc} & \begin{array}{ccc} q_1 & q_2 & q_3 \end{array} \\ H = \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 1 & 4 & 10 \\ .25 & 1 & 4 \\ .1 & .25 & 1 \end{bmatrix} \end{array} \rightarrow \begin{array}{cc} & \begin{array}{ccc} q_2 & q_3 & q_4 \end{array} \\ \begin{array}{c} q_1 \\ q_2 \\ q_3 \end{array} & \begin{bmatrix} 1 & 4 & 6.7 \\ .25 & 1 & 4 \\ .15 & .25 & 1 \end{bmatrix} \end{array}$$

The second expert assigns another confirmation space, since he considers a lower value for buckling – support settlement.

$K = (0.9/0.4/0.1) \rightarrow (0.8/0.5/0.2)$

With the correction vector $K = (0.8/0.5/0.2)$, $V_1 = (0.7/0.9/0.3)$ and the normalized weighting vector $W^* = (0.64/0.29/0.08)$ one should find out first if the damage cause V_1 still is dominant (equation 75):

$$d_w(V_1/V_2) = 0.64 \times (0.7 - 0.3) + 0.29 \times (0.9 - 0.9) + 0.08(0.3 - 0.1) = 0.272$$

The obtained results shows that V_1 is relevant, but to a lower extent

$$d_w(V_1/V_3) = 0.64 \times (0.7 - 0.1) + 0.29 \times (0.9 - 0.1) + 0.09(0.3 - 0.9) = 0.67$$

V_1 is still relevant, but to a lower extent compared to the original evaluation.

The new confirmation degree for cause no. 1 results in

$$C_{V1} = (0.64 \times 0.8)0.7 + (0.29 \times 0.5)0.9 + (0.08 \times 0.2)0.3 = 0.494$$

V_1 is relevant with a probability of only 49 % compared to the first evaluation, where it was 65 % (buckling).

The verification with respect to the beam, column 1 and 2 and shows that the opinion differences between the two experts do not extensively change the calculation results. So, one may assume that the fuzzy algorithm is sufficiently robust and reliable. The remaining calculations are performed with the result from expert 1. In the same way as already is described in section 4.4, the obtained fuzzy values are retranslated in linguistic values in order to take back to approximate consequences of the original premises. This defuzzification process ends up in damage indices D_i to be assigned weights w_i according to their local influence on component or system performance.

The results obtained from a calculation with singletons can be directly translated in human language, since the confirmation degree C already represents a kind of understandable solution (Tilli, Th.; 1995). C does not guarantee an explicit information such as, that the damage cause V_i or V_j alone is decisive. However, it represents a degree of confidence as is already expressed by its name. It participates in a weighted vote to come to an overall conclusion, where the weight is obtained heuristically. A low confirmation degree should be justified by additional information, especially if there are symptoms of unresolved controversies. Some failure modes may have more severe consequences than others.

So, is e.g. “joint disruption worse than beam hinging”. The weighted arithmetic mean value D_w can then be calculated dependent on all failure modes, i.e. dependent on the chronological order of the damage causes V_i deduced from the weighted Hamming distance d_w. Significant participants are emphasized using weights w_i according to the opinion of the decision maker:

$$D_w = \frac{D_i^2 w_i}{\sum D_i^2} \quad \text{(equation 77)}$$

Type, significance and reliability or truth of the obtained information may additionally be considered by dividing D_i with coefficients γ_i (see Table 4-6). The actual computation process will be shown later in the example problems (three span girder bridge and protective structure).

$$\eta = \frac{(D_i / \gamma)^2 \times w_i}{\sum D_i^2} \quad \text{(equation 78)}$$

Dependent on type, significance and reliability of the obtained information, the damage extent is $\eta = \frac{(D_i / \gamma_i)^2 \times w_i}{\sum D_i^2}$. An assignment of weights to the damage indices can be performed using C and the chronological order of the damage causes.

Superstructure

The weighted damage index results in (equation 78)

$$\eta = \frac{(D_M/\gamma)^2 w_M + (D_F/\gamma)^2 w_F + (D_S/\gamma_S)^2 w_S (D_{T-C}/\gamma_{T-C})^2 w_{T-C} + (D_{V-C}/\gamma_{V-C})^2 w_{V-C} + (D_{H-C})^2 w_{H-C} + (D_{D-C})^2 w_{D-C} + (D_D)^2 w_D}{D_M^{\ 2} + D_F^{\ 2} + D_S^{\ 2} + D_{T-C}^{\ 2} + D_{V-C}^{\ 2} + D_{H-C}^{\ 2} + D_{D-C}^{\ 2} + D_D^{\ 2}}$$

$$= \frac{(0.3)^2 \times 0.9 + (0.1/0.6)^2 \times 0.3 + (0.6)^2 \times 0.6 + (0.2)^2 \times 0.3 + (0.2)^2 \times 0.6 + (0.2)^2 \times 0.2 + (0.3)^2 \times 0.6 + (0.1)^2 \times 0.6}{0.3^2 + 0.1^2 + 0.6^2 + 0.2^2 + 0.2^2 + 0.2^2 + 0.3^2 + 0.1^2}$$

$$= \frac{0.081 + 0.008 + 0.216 + 0.012 + 0.024 + 0.008 + 0.054 + 0.06}{0.68}$$

$$= 0.68$$

The obtained result helps assign the superstructure in a condition state using the membership functions of Figure 5-8 (relevant is the coloured one). It belongs to both, “moderate” and “considerable” again reflecting the vague notion of human thinking:

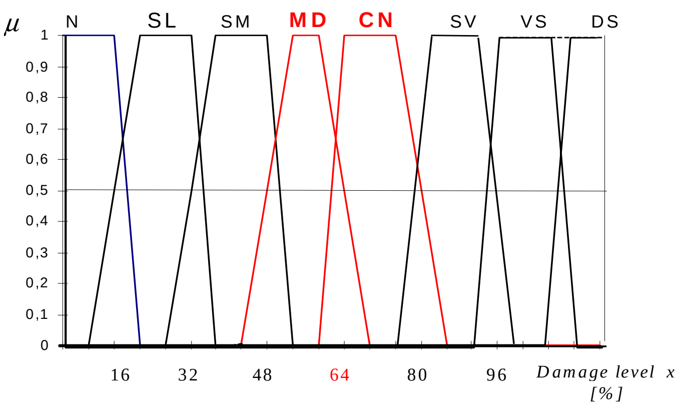

Figure 5-8: Membership Functions

With a damage extent of $\eta = 0.68$ (equation 78), an element importance of $I = 1.0$ and a prognostic factor $L = 0.8$ (rather probable) the risk of the superstructure results in (equation 55):

$p_{superstructure} = \eta^I L = 0.68^{1.0} \times 0.8 = 0.544$, actually a relatively low risk compared to that one of the other structural components.

Beam

The damage extent results in results in (equation 78)

$$\eta = \frac{(D_{Reduced\,Strength} / \gamma_{R-S})^2 w_{R-S} + (D_{SectionLoss} / \gamma_{S-L})^2 w_{S-L} + (D_{Cracks} / \gamma_C)^2 w_C + (D_{Excessive\ \ Deflection} / \gamma_{E-D})^2 w_{E-D}}{D_{R-S}{}^2 + D_{S-L}{}^2 + D_C{}^2 + D_{E-D}{}^2}$$

$$\eta = \frac{(0.1/0.8)^2 \times 0.05 + (0.2/0.6)^2 \times 0.1 + (0.7/0.8)^2 \times 0.4 + (0.9/0.8)^2 \times 0.45}{0.1^2 + 0.2^2 + 0.7^2 + 0.9^2} = 0.66$$

The beam may be assigned with "considerable":

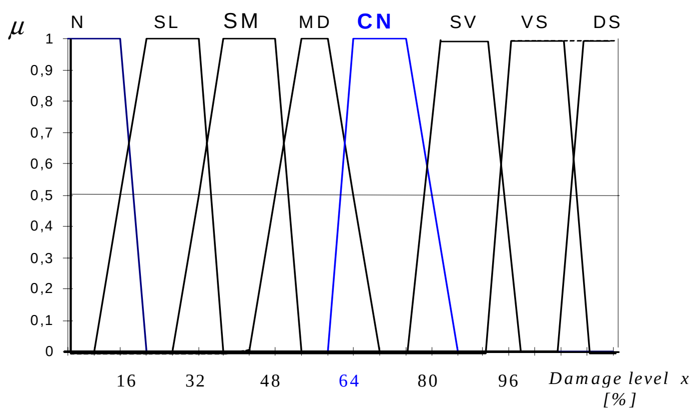

Figure 5-9: Membership Functions

With a damage extent of $\eta = 0.66$ (see Figure 5-9 with its coloured membership function), an element importance of $I = 1.0$ and a prognostic factor of $L = 0.4$ (rather improbable) the risk $p_i = \eta^I L$ (equation 55) of the beam is $p_{girder} = 0.66^{1.0} \times 0.4 = 0.264$, is relatively low compared to that of column 1 or 2.

Column 1

The damage extent results in (equation 78)

$$\eta = \frac{(D_{Loss-Support} / \gamma_{L-S})^2 w_{L-S} + (D_{Joint-Failure} / \gamma_{J-F})^2 w_{J-F} + (D_{Buckling} / \gamma_B)^2 w_B}{D_{L-S}{}^2 + D_{J-F}{}^2 + D_B{}^2}$$

$$= \frac{(0.9/1)^2 \times 0.8 + (0.3/0.6)^2 \times 0.1 + (0.1/0.8)^2 \times 0.1}{0.9^2 + 0.3^2 + 0.1^2} = 0.91$$

It is assigned to the condition state “severe” and “very severe”.

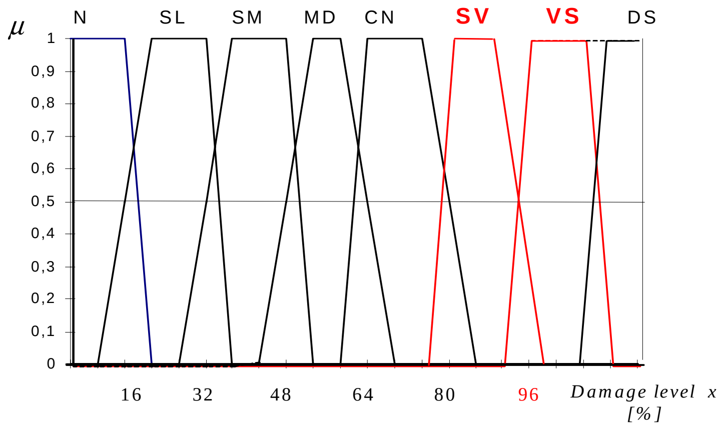

Figure 5-10: Membership Functions

With a damage extent of $\eta = 0.91$ (see Figure 5-10 with its coloured membership function), an element importance of $I = 0.9$ (the importance may be assigned 10 % less than for column 2 because human life is not threatened due to warning and a prognostic factor of $L = 0.6$ (rather probable) the risk of $p_i = \eta^I L$ (equation 55) of column 1 results in $P_{columnn\,1} = 0.91^{0.9} \times 0.6 = 0.55$, actually a relatively high risk.

For **Column 2** the damage extent results in (equation 78)

$$\eta = \frac{(0.2/1)^2 \times 0.1 + (0.5/0.6)^2 \times 0.2 + (0.9/1.0)^2 \times 0.7}{0.2 + 0.5 + 0.9} = 0.70$$

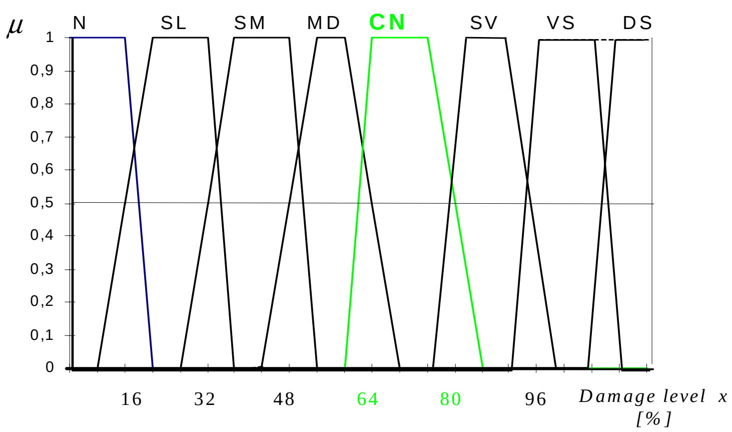

Figure 5-11: Membership Functions

The structure is assigned with “Considerable”. With a damage extent of $\eta = 0.70$ (see Figure 5-11 with its coloured membership function), an element importance of $I = 1.0$ and a prognostic factor of $L = 0.9$ (very probable) results in (equation 55) $p = \eta^{I} L = p_{columnn\,1} = 0.70^{1.0} \times 0.9 = 0.63$, an extremely high risk.

The calculations for the risk of the whole system can be performed as described in chapter 4.3. The three span girder represents a serial-connected determinate system (if one column fails the beam fails due to overstress -see Figure 5-12) with a correlation coefficient <0 (stochastic independent). Its risk can be calculated with (equation 58)

$$1 - \prod_{i=1}^{n} \left(1 - p_i\right) < P_f < \left(\sum_{i=1}^{n} p_i\right)$$

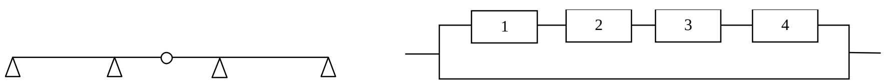

Figure 5-12: Three Span Bridge represents a Serial Connected System

With $p_{sup\,erstructure} = 0.544$ $p_{beam} = 0.264$, $p_{column\,1} = 0.55$ and $p_{column\,2} = 0.63$ the risk of the whole system results in

$$1 - \left(1 - p_1\right)\left(1 - p_2\right)\left(1 - p_3\right)\left(1 - p_4\right) < P_f < p_1 + p_2 + p_3 + p_4$$

$$1 - \left(1 - 0.544\right)\left(1 - 0.264\right)\left(1 - 0.55\right)\left(1 - 0.63\right) < P_f < 0.544 + 0.264 + 0.55 + 0.63$$

$$1 - 0.456 \times 0.736 \times 0.45 \times 0.37 < P_f < 1.988$$

$$P_f = 0.944$$

Risk of the whole system results in 94 %.

The next theoretical example (see Figure 5-13) represents a protective structure composed of wall 1 and 2, both assumed to be suffering from perforation or cratering damage, a foundation slab mainly damaged due to water infiltration, and four steal cables with a risk of $p_1 = 0.01$ (damage extent $\eta_{Steel} = 0.03$, Importance $I_{Steel} = 0.5$ and prognostic factor $L_{Steel} = 0.5$), each.

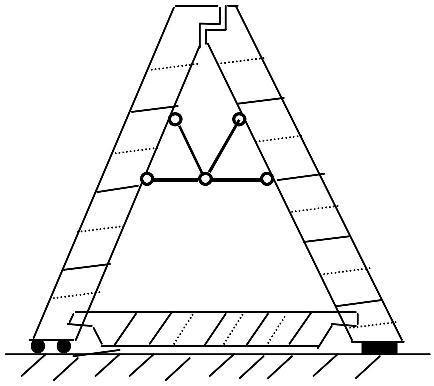

Figure 5-13: Protective Structure

Since for wall 1 and 2 we are not sure about η required to calculate component risk with $p_i = \eta^I L$, the procedure based on fuzzy-logic will be applied. Just as in example 1 the matrix R and K have been fuzzified in dependence of the results from visual, non-destructive or laboratory investigations. The singletons μ_{ij} and k_i values define the cause-effect relationship of the characteristics and the probability that a certain failure mode actually is exhibited, respectively. The matrix H intends to describe the relative importance of the damage effects. Table 5-10 shows the relationship between cause and effect for wall 1and 2:

	Insufficient Protective Cover	*Diagonal Tension / Shear*	*Flexure*	*Concrete-Rebar Interaction*	*Impul-sive Loading*	Corrosion
Spalling	*0.9*	*0.6*	*0.6*	*0.9*	*0.9*	0.7
Rotation Slab-Wall	*0*	*0.2*	*0.8*	*0*	*0.3*	0
Miscel-lanous Cracks	*0.9*	*0.2*	*0.5*	*0.9*	*0*	0.6
Deflection	*0*	*0*	*0.9*	*0*	*0.9*	0

Table 5-10: Relationship between Damage Cause and Effect

$$
R = \begin{array}{c} \\ q_1 \\ q_2 \\ . \\ q_n \end{array}
\begin{array}{c}
\begin{array}{cccc} V_1 & \cdots & & V_n \end{array} \\
\begin{bmatrix} \mu_{11} & . & . & \mu_{1n} \\ \mu_{21} & . & . & \mu_{2n} \\ . & . & . & . \\ \mu_{n1} & . & . & \mu_{nn} \end{bmatrix}
\end{array}
\qquad \text{(equation 71)}
$$

$$
R = \begin{array}{c} \\ q_1 \\ q_2 \\ q_3 \\ q_4 \end{array}
\begin{array}{c}
\begin{array}{cccccc} V_1 & V_2 & V_3 & V_4 & V_5 & V_6 \end{array} \\
\begin{bmatrix} 0.9 & 0.6 & 0.6 & 0.9 & 0.9 & 0.7 \\ 0 & 0.2 & 0.8 & 0 & 0.3 & 0 \\ 0.9 & 0.2 & 0.5 & 0.9 & 0 & 0.6 \\ 0 & 0 & 0.9 & 0 & 0.9 & 0 \end{bmatrix}
\end{array}
$$

The vectors for the damage causes for the protective structure are

$$
\begin{aligned}
V_1 &= (0.9/0/0.9/0) \\
V_2 &= (0.6/0.2/0.2/0) \\
V_3 &= (0.6/0.8/0.5/0.9) \\
V_4 &= (0.9/0/0.9/0) \\
V_5 &= (0.9/0.3/0/0.9) \\
V_6 &= (0.7/0/0.6/0)
\end{aligned}
$$

Arbitrary combination of a shear and flexure mode represents a conflict resolution, which implies that there is no overwhelming evidence to substantiate one of the two damage modes (retrieving info is very difficult). We do not differentiate all the different types of corrosion such as concrete-corrosion due to physico-chemical disintegration and rebar-corrosion due to electrochemical deterioration. The relative importance of the damage effects is shown in Table 5-11:

	Spalling	*Rotation Slab-Wall*	*Miscellanous Cracks*	*Deflection*
Spalling	*1*	*4*	*4*	4
Rotation Slab-Wall	*0.25*	*1*	*0.2*	1
Miscellanous Cracks	*0.25*	*5*	*1*	5
Deflection	*0.25*	*1*	*0.2*	1

Table 5-11: Relative Importance of the Damage Effects

$$H = \begin{bmatrix} 1 & 4 & 4 & 4 \\ 0.25 & 1 & 0.2 & 1 \\ 0.25 & 5 & 1 & 5 \\ 0.25 & 1 & 0.2 & 1 \end{bmatrix} \qquad \text{(equation 73)}$$

The weighted Hamming distance (equation 75) is calculated in dependence of

$$\begin{aligned} & W^* = (13 / 2.45 / 11.25 / 2.45) \\ & \sum 29.15 \qquad \text{(equation 74)} \\ & normalized \ W^* = (0.45/0.08/0.39/0.08) \end{aligned}$$

$$d_w(V_1 / V_2) = 0.45(0.9 - 0.6) + 0.08(0 - 0.2) + 0.39(0.9 - 0.2) + 0.08(0 - 0) = 0.39$$

$$d_w(V_1 / V_3) = 0.45(0.9 - 0.6) + 0.08(0 - 0.8) + 0.39(0.9 - 0.5) + 0.08(0 - 0.9) = 0.16$$

$$d_w(V_1 / V_4) = 0.45(0.9 - 0.9) + 0.08(0 - 0) + 0.39(0.9 - 0.9) + 0.08(0 - 0) = 0$$

$$d_w(V_1 / V_5) = 0.45(0.9 - 0.9) + 0.08(0 - 0.3) + 0.39(0.9 - 0) + 0.08(0 - 0.9) = 0.255$$

$$d_w(V_1 / V_6) = 0.45(0.9 - 0.7) + 0.08(0 - 0) + 0.39(0.9 - 0.6) + 0.08(0 - 0) = 0.207$$

Both damage causes, insufficient protective cover V_1 and insufficient concrete-rebar interaction V_4 are equally relevant. The chronological order of the other damage causes can be deduced from the d_w values.

$$\begin{matrix} V_1 \\ V_4 \end{matrix} \rightarrow V_3 \rightarrow V_6 \rightarrow V_5 \rightarrow V_2 .$$

Dependent on the correction vector $K = (0.8 / 0.2 / 0.8 / 0.4)$ and the damage causes V_1 / V_4 the confirmation degree for cause no. 1 and cause no. 4 results in the confirmation degree (equation 76)

$$C_{V1} = C_{V4} = (W^* \times K)V_{1/4}$$
$$= 0.45 \times 0.8 \times 0.9 + 0.08 \times 0.2 \times 0 + 0.39 \times 0.8 \times 0.9 + 0.08 \times 0.4 \times 0$$
$$= 0.6$$

The damage extent η can be calculated (γ is supposed to be 1.0 since, due to the significance of a protective structure, the assessment is performed very much in detail, so that the damage indices do not have to be reduced) dependent on the damage indices D_i with equation 77 / 78:

$$\eta = D_w = \frac{D_S^{\ 2} \times w_S + D_R^{\ 2} \times w_R + D_C^{\ 2} \times w_C + D_D^{\ 2} \times w_D}{D_S + D_R + D_C + D_D}$$
$$= \frac{0.7^2 \times 0.8 + 0.1^2 \times 0.9 + 0.7^2 \times 0.4 + 0.3^2 \times 0.7}{0.7 + 0.1 + 0.7 + 0.3}$$
$$= 0.66$$

Both, wall 1 and wall 2 are assumed to have the same characteristics with

$$\eta_{wall-1} = \eta_{wall-2} = 0.66$$
$$I_{wall-1} = I_{wall-2} = 1.0$$
$$L_{wall-1} = L_{wall-2} = 0.55$$
$$p_1 = p_2 = p_{wall-1} = p_{wall-2} = \eta^I L = 0.66^{1.0} \times 0.55 = 0.36$$

One could perform the fuzzy calculations also for the foundation slab, but here there are assumed only two damage causes and two damage effects. Dependent on the damage indices for spalling and material breakdown the damage extent for the foundation slab can be calculated with (equation 77 / 78):

$$\eta_{Found} = \frac{D_S^{\ 2} \times 0.8 + D_B^{\ 2} \times 0.3}{D_S^{\ 2} + D_B^{\ 2}}$$
$$= \frac{0.6^2 \times 0.8 + 0.3^2 \times 0.3}{0.6 + 0.3} \qquad \text{(equation 77 / 78)}$$
$$= 0.7$$

$$\eta_{Found} = 0.7$$
$$I_{Found} = 1$$
$$L_{Found} = 0.3$$
$$p_3 = p_{Founde} = \eta^I L = 0.21$$

The steel cables are very well fabricated. Their risk may therefore assumed to be quite low with $p_4 = p_5 = p_6 = p_7 = 0.01$. In this example (see Figure 5-14), the elements *1*, *2* and *3* are connected in a serial manner with equation 57: ($\max_{i=1}^{n} p_i < P_f < 1 - \prod_{i=1}^{n} (1 - p_i)$). Here, the element *3* can be replaced by a composition of serial-connected elements *4-5-6-7* to participate with equation 60 as a parallel system $\prod_{i=1}^{n} p_i < P_f < \min_{i=1}^{n} p_i$.

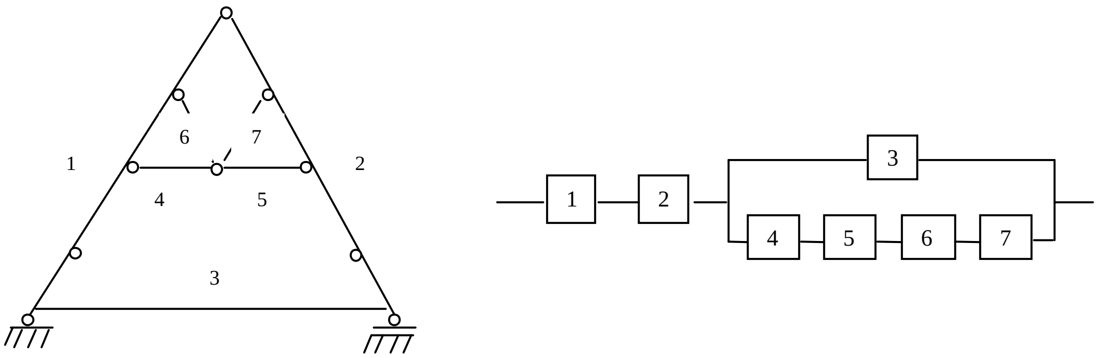

Figure 5-14: Structural System with Serial and Parallel Connected Components

So, with $p_{wall-1} = p_{wall-2} = 0.36$, $p_{Found} = 0.21$ and $p_4 = p_5 = p_6 = p_7 = 0.01$ risk for the whole structure results in (combination of equation 57 and 60):

$$
\begin{aligned}
P_f &= 1 - \prod_{i=1}^{2} (1 - p_{fi}(DE_i)) \times \left\{1 - p_{f3}(DE_3)\left[1 - \prod_{i=4}^{7} (1 - p_{fi}(DE_i))\right]\right\} \\
&= 1 - (1 - 0.36)^2 \times \left\{1 - 0.21\left[1 - \prod_{i=4}^{7} (1 - 0.01)\right]\right\} \\
&= 1 - (1 - 0.72 + 0.13)\{1 - 0.21[1 - 0.961]\} \\
&= 1 - 0.41 \times 0.992 \\
&= 0.59
\end{aligned}
$$

Even though the risk of the steel cables is assumed to be quite low with 0.01 the protective structure has a risk of 59 %.

6 Summary and Recommendations

Whereas the design of new structures is almost completely regulated by codes, there are no objective ways for the evaluation of existing facilities. Experts often are not familiar with the new tasks in system identification and try to retrieve at least some information from available documents. They therefore make compromises which, for many stakeholders, are not satisfying. Consequently, this publication presents a more objective and more realistic method for condition assessment. Necessary basics for this task are fracture mechanics combined with computational analysis, methods and techniques for geometry recording and material investigation, ductility and energy dissipation, risk analysis and uncertainty consideration.

Since micro-cracking, crack-coalescence or -nucleation and macro-crack formation can -dependent on fracture toughness- change a ductile failure mode into a brittle one, fracture mechanics is a research area not only important for design. Appropriate computer tools dealing with both, micro- and macro- damage have to be selected considering the type of loading (static, dynamic or impulsive) and the specific failure mode characteristics (chapter 2.2). After having identified system geometry including dimensions of key components, strength, aggressive agents, cracks etc. are explored using destructive and nondestructive tests (chapter 2.3). Ductility and energy absorption have been dealt with in a separate section, since they may significantly delay system collapse. Since the cross section at some selected areas does not inform about instability after hinge formation, the extent of balance between resistance and ductility of all structural components, of their joints and of the system as a whole has to be verified (chapter 2.4).

Deterministic tools such as strain or energy measurement need experienced engineers knowing type and location of the failure mode. However, damage is a function of statistical correlation between varying load and resistance parameters. Different loading conditions always lead to a different failure sequence. Therefore, probabilistic analyses have been used to capture possible load paths and scenarios, or to predict an adverse event with its effects on the load-bearing capacity. Risk of structural failure does not only depend on the damage extent, but also on its likelihood of occurrence expressed by the prognostic factor and on relative consequences in turn related to the vulnerability and importance of a structure. Therefore risk analysis is defined as the detection of weak areas or faults and as the activity of investigating system topology with respect to possible failure consequences. It constitutes an important part in damage quantification and helps preserve human life together with economic values (chapter 2.5).

Uncertainty exists -among other reasons- due to measurement errors, due to general stochasticity inherent in laboratory tests and due to human errors. Epistemic (nonrandom) uncertainty often being larger than aleatory (random) uncertainty can lead an engineer into a false direction. Random errors can be dealt with by statistical methods relying on absolute values. This is not true for non-random errors therefore requiring a different approach. Subjective judgement is not sufficient for decision-making, especially when the penalty for an error is severe (chapter 2.6).

Present tools for evaluation perform research on how to analytically conceptualize a structure directly from given loads and measured response. Since defects are not necessarily visible or in a direct way detectable, several damage indices are combined and integrated in a model of the real system (chapter 3.1). At this stage of analysis, data preparation is of crucial importance in which specialized signal processing techniques are used to efficiently transform the measurement data in a useful database content and format. Compared to common parametric techniques only estimating damage parameters, nonparametric neuronal networks are capable to learn by directly relating input and output parameter (chapter 3.2). In the course of modeling engineers idealize the structure considering the peculiarities of existing infrastructure. Model updating consists of verification (is the system built right?), validation (was the right system built?) and accreditation (is it appropriate and accepted by the user? -chapter 3.3).

In order to successfully solve some of the problems coming from deficiencies in present evaluation procedures the author proposes a holistic methodology additionally applying cause-effect analyses combined with fuzzy-logic, sensitive analyses and hazard analyses. Civil engineering is not an exact science itself, but an art using scientific knowledge or subjective assumptions and therefore often is referred to as an “applied science”. Engineers therefore can never guarantee correctness, though their experience may guide them in choosing the most relevant damage indices (earthquake damaged buildings are e.g. dealt with by the indices “maximum story drift”, “minimum frequency in either direction” and “change of frequency”). The lack of prototype testing facilities does not avoid the public to require high levels of safety. People desire to have eliminated uncertainty completely and think, that it is not sensible to expend large efforts to reduce some sources of risk to vanishingly small levels, while others go unaddressed (chapter 4.1).

Fuzzy-sets are ideally suited to illustrate parametric/data uncertainty and system- or model uncertainty. Trapezoidal membership functions may very well represent the condition state of structural components as function of damage extent or performance (chapter 4.2.1). Tthe residual load-bearing capacity can be determined by successively performing analyses in three steps according to chapter 4.2.2, 4.2.3 and 4.2.4. The “Screening assessment” shall eliminate a large majority of structures from detailed consideration and advise on immediate precautions to save lives and high economic values. Here, the defects have to be explicitly defined and located (chapter 4.2.2). If this is impossible, an “approximate evaluation” should follow describing system geometry, material properties and failure modes in detail. Here, a fault-tree helps investigate defaults in a systematic way avoiding random search or negligence of important features or damage indices. In order to inform about the structural system it is deemed essential not only due to its conceptual clarity, but also due to its applicational simplicity (chapter 4.2.3). It therefore represents an important prerequisite in condition assessment though special circumstances might require “further investigations” to consider the actual material parameters and unaccounted reserves due to spatial or other secondary contributions. Here, uncertainties with respect to geometry, material, loading or modeling should in no case be neglected, but explicitly quantified (chapter 4.2.4).

Postulating a limited set of expected failure modes is not always sufficient, since detectable signature changes are seldom directly attributable and every defect might -together with other unforeseen situations- become decisive. So, a determination of all possible scenarios to consider every imaginable influence would be required. Risk is produced by a combination of various and ill-defined failure modes. Due to the interaction of many variables there is no simple and reliable way to predict which failure mode is dominant. Risk evaluation therefore comprises the estimation of the prognostic factor with respect to undesirable events L_i, component importance I_i and the expected damage extent η (chapter 4.3).

In case of multi-state material characteristics or in case of uncertainties in geometry, loads and modeling singletons within a fuzzy matrix or vector may serve for the calculation indicating dominant failure modes. Objective results may be obtained even if the specialist is not sure about type and location of defects. Though an assignment of the singletons per se is subjective, the procedure guarantees, that not already in the beginning important attributes are neglected just because their influence is considered to be low, or because few information is available (chapter 4.4). The proposed methodology is illustrated in chapter 5 using two structural types of system, namely a well defined line structure, a three-span bridge, and a protective structure.

The purpose of this publication is to introduce the civil engineering community to a new perspective for condition assessment. The advantage of the proposed procedure is clear given the more detailed, verifiable, conclusive and legitimate results which can be generalized for an application in various structural systems. The approach provides a logical way of resolving even conflicting statements and, of combining data from various sources. It might be argued by some engineers that by the use of subjectively derived fuzzy relations objectivity is lost and that extended research would be capable of providing better data and more understanding. However, recall that the objective here is to illustrate, how multi-state failure mechanisms can be dealt with, and to show corresponding potential conclusions that could be arrived at under various and difficult circumstances. Although many encouraging, sensible ideas have already been developed, the remaining difficulties associated with fuzzification and defuzzification emphasize the importance of further research activities. Also, conventional expert systems still suffer from a lack of consense among experts and from vagueness in their descriptions. Fuzzy-expert systems could work with less rules and therefore dispose of a better transparency, avoid inconsistency and incompleteness.

So, future research could pursue three directions: first, combinations of several features could be subjected to both, selection and extraction activities in order to discover better ones. Second, relational data with values assigned by experts could be derived to propose additional fuzzy-algorithms. Third, a combination of fuzzy-logic and neuronal networks may enhance the range of utilization with respect to both tools and increase the efficiency of calculation procedures considerably. On the one hand, neuronal networks could augment numerical processing by generating, adapting or tuning fuzzy-sets. On the other hand, imprecise knowledge formalized in membership functions could be applied in training-sets to augment the interpretation or learning capabilities of neuronal networks and to describe results from calculations which, actually are still characterized by the black-box phenomena implying that the user cannot replicate the train of thought or calculation procedure. By translating neuronal networks into a fuzzy-set, its black-box characteristics would no more represent a problem, since they can be interpreted as a fuzzy-set.

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1 Einführung

Schon immer galt es, Bauwerke nach Katastrophen auf ihre Tragfähigkeit zu untersuchen. Früher versetzten Naturereignisse wie Erdbeben, Flut oder Feuer die Bevölkerung in Schrecken. Heute muß man zusätzlich mit einem Terroranschlag oder Flugzeugabsturz rechnen. So müssen Ingenieure außer der Bemessung von Gebäuden zusätzliche Aufgaben übernehmen. Sie unterstützen Bauherren bei der Entscheidung, ob ein geschädigtes Bauwerk abgerissen werden muß, oder es noch teilweise nutzbar ist. Diese komplexe Fragestellung fordert eine koordinierte Zusammenarbeit verschiedener Spezialisten, aber auch von Generalisten mit vertieftem Wissen über Zusammenhänge. Mehrere Fachdisziplinen müssen für die Zustandsanalyse vernetzt werden (siehe Abbildung 1).

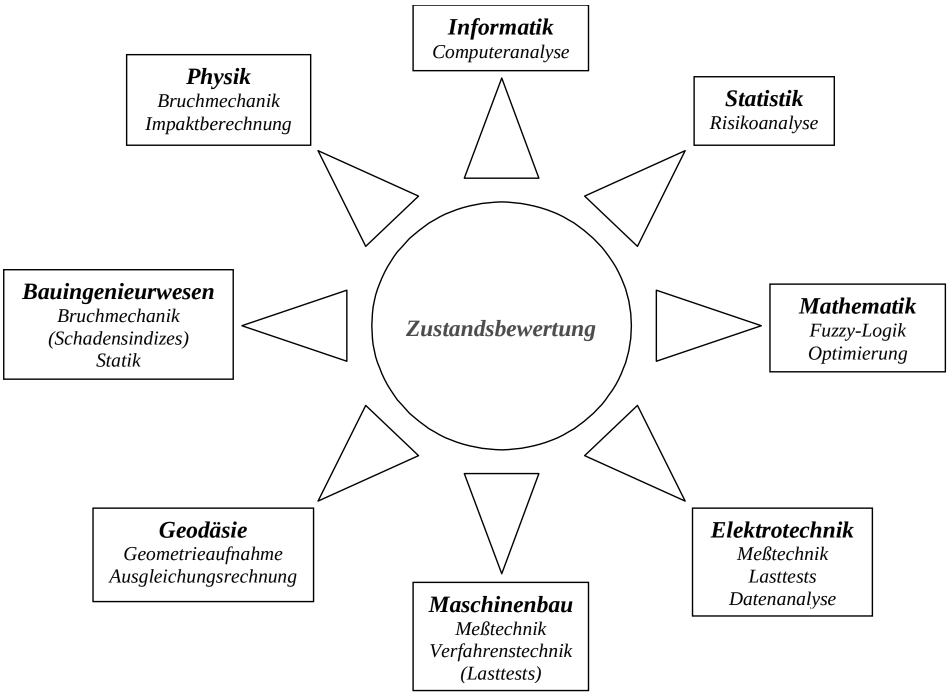

Abbildung 1: Fachdisziplinen für die Zustandsanalyse

1.1 Ausgangssituation

Sowohl die theoretischen Grundlagen der Schadensanalyse, als auch Methoden und Techniken für die Geometrie- und Materialerkundung sind in den letzten Jahrzehnten ausgiebig erforscht und verbessert worden. In etwas geringerem Umfang trifft dies auch zu für Themen wie Duktilität, Energieabsorption, Risikoanalyse und Beschreibung der Unsicherheit (siehe Kapitel 2.1 bis 2.6). Das gesammelte Potential an Wissen ist zur Entwicklung von Bewertungsmethoden für Bauwerksschäden genutzt worden. Die aktuelle Untersuchungsmethode läuft nach einem geregelten Schema ab: Nach einer Definition von Schadenindizes erfolgen Datenbearbeitung bzw. -analyse und Modellierung. Für die Modellverbesserung sind bislang mehrere Verfahren, insbesondere die Neuronalen Netze, erprobt worden (siehe Kapitel 3).

1.2 Problemstellung und Motivation

Bei der Entwicklung bislang verwendeter Verfahren ging man davon aus, daß ein Bauwerk gerade errichtet wird und somit der Datenfluß kontinuierlich, vollständig und unmißverständlich erfolgt (vorgegebenes statisches System unter bekannter Belastung). Bei der Bewertung eines bestehenden Bauwerks muß die Systemstruktur abhängig von der aufgebrachten Belastung anhand seiner Reaktion identifiziert werden. Beide Aufgaben sind verschieden und können daher nicht gleich behandelt werden.

1.3 Zielsetzung

Bauherren sind mit der momentanen Zustandsbewertung basierend auf subjektiv ausgewählten Schadenbildern oft unzufrieden. Aus Gründen der Sicherheit und Wirtschaftlichkeit fordern sie nachvollziehbare, objektivere Methoden und erwarten, daß Unsicherheiten nicht vertuscht, sondern explizit in die Rechnung einbezogen werden. Bestehende Mängel in Analysen legen dringende Veränderungen in einem, für Ingenieure ungewohnten Bereich, nahe. Diese Veröffentlichung soll aufzeigen, wie mögliche Versagensmechanismen systematisch untersucht, sowie Details nachvollzogen werden können. Im Rahmen einer ganzheitlichen Methodik erfolgt hiermit in drei Stufen eine Grobeinstufung, abschätzende Beurteilung und weitergehende Untersuchung. Während sich die Grobeinstufung mit einer raschen Analyse des wesentlichen Tragverhaltens begnügt, hat die abschätzende Beurteilung eine genauere Strukturbewertung zum Ziel. Ist dies aufgrund von komplexen Versagensmechanismen nicht möglich, wird in einer weitergehenden Untersuchung abgeklärt, wie man mit dem Bauwerk weiter verfährt, um eindeutig über Abriß oder Nutzung zu entscheiden. Kann nämlich die Beziehung zwischen Ursache und Wirkung von Schäden anschaulich dargestellt und quantifiziert werden, wird eine Einstufung in Qualitätsklassen möglich.

2 Grundlagen zur Beurteilung von Bauwerken

2.1 Einführung

Eingeführt wird in die, für eine Bauwerksanalyse erforderlichen Fachdisziplinen. Hierfür werden die wichtigsten theoretischen Grundlagen der Schadensanalyse, der Geometrie- und Materialaufnahme, der Duktilität und Energieabsorption, der Risikoanalyse und Kostenminimierung sowie der Berücksichtigung von Unischerheitenkurz erläutert.

2.2 Theoretische Grundlagen der Schadensanalyse

Die Bruchmechanik bezeichnet den Bereich der Festkörpermechanik und Materialtechnik, der sich mit dem Rißverhalten beschäftigt. Sie setzt globale Kräfte und Deformation mit dem lokalen Bruchverhalten an einer Rißspitze in Beziehung, um das mechanische Verhalten geschädigter Bauteile zu beschreiben. Die Bruchmechanik unterscheidet Makro-, Meso- und Mikroschädigungen und veranschaulicht sie mit 2D- und 3D-Modellen. Um die Neuverteilung von Lasten nach Rißbildungen zu ermitteln, untersucht sie alle Tragglieder gleichzeitig (Karihaloo, B.; 1996). Demnach ist bei feinen, homogen verteilten Rissen, die Steifigkeit gleichmäßig (elastisch) und bei etwas breiteren Rissen differenziert (elasto-plastisch) zu vermindern, während große

Risse diskret zu modellieren sind. Abhängig von den spezifischen Materialeigenschaften und der Belastung gehen die meisten Computercodes jedoch von einer gleichmäßigen Schädigung aus, ohne Topologie, Lastgeschichte oder den Einfluß lokalen Versagens auf das globale Tragverhalten zu berücksichtigen. Da verschiedene Schadensereignisse jeweils ein unterschiedliches Strukturverhalten bewirken, sind sie explizit zu erfassen. Statische und dynamische Beanspruchungen erfordern andere Analyseverfahren als impulsartige. Für Explosionsberechnungen reichen materialunabhängige Formulierungen in Form von Masse-, Impuls- und Energieerhaltung nicht aus. Die, bei einer Explosion entstehende extrem hohe Druckbelastung verändert die Materialdichte von Beton ganz erheblich. Bei hoher Dehngeschwindigkeit nimmt seine Festigkeit zu. Um Tragreserven durch Materialduktilität und Hysteresis erfassen zu können, sollte die Veränderlichkeit der Festigkeit in einem Materialmodell berücksichtigt werden.

2.3 Geometrie- und Materialerkundung

Da in Computeranalysen das Tragsystem wirklichkeitsgetreu abgebildet werden muß, sind Geometrie und Material vorher zu erkunden. Die Abmessungen und Lage aller Tragglieder, Verformungen oder etwaige Schiefstellungen werden z.B. mit Hilfe der Tachymetrie oder der Photogrammetrie (terrestrische und luftbildgestützte) ermittelt. Daneben sind auch Handaufmaß, GPS, reflektorbezogene und reflektorlose Verfahren üblich. Computergestützte 2D und 3D Scanner werden seit geraumer Zeit im Hoch- und Tiefbau benutzt. Die Videotechnik dient der Dokumentation bei der Erfassung, wobei die Geometrie des Tragwerks anhand verschiedener Bildfolgen automatisch rekonstruiert werden kann (spezielle Software entzerrt die Bilder orthogonal). Zusammen mit einer maßlichen Analyse läßt sich so ein Gebäudemodell generieren.

Für eine intelligente Vorgehensweise im Rahmen der Geometrie- und Materialerkundung sind noch weitere Erfassungstechnologien zu erproben. Optimal wäre es, wenn parallel zur Geometrie auch Material und Bauzustand erfaßt werden könnten. Nach einer Plausbilitätsprüfung ließen sich so der Geometrie (Lage und Zuordnung der Bauteile) beispielsweise Rißbildung und Oberflächenbeschaffung gegenüberstellen. Zerstörende und zerstörungsfreie Tests informieren dann genau über Bewehrungslage und -durchmesser, Betondeckung, Betonfestigkeit bzw. -zusammensetzung (Korrosionsgefahr durch aggressives Milieu oder Feuchtigkeit) und Rißgeometrie.

2.4 Duktilität und Energieabsorption

Duktilität und Energieabsorption werden in einem eigenem Abschnitt behandelt, weil sie ein Versagen der Gesamtstruktur wesentlich verzögern können. Deshalb lohnt es sich auch bei jedem Querschnittsversagen zu prüfen, ob das entsprechende Tragglied und angrenzende Knotenpunkte duktil ausgebildet sind. Ein einzelner deterministischer Spannungsnachweis informiert keinesfalls über beginnende Instabilität des gesamten Systems durch Fließgelenkbildung. Die Systemtopologie bestimmt maßgeblich, wie vertikale und horizontale Lasten abgetragen werden und, wie Energie absorbiert wird. Torsion durch zu großen Abstand zwischen Masseschwerpunkt und Lage von Bauteilen hoher Festigkeit, einspringende Ecken (Grundriß) oder unterschiedliche Geometrie aufeinanderfolgender Stockwerke können eventuell noch vorhandene

Tragreserven erschöpfen. Die Robustheit, d.h. Tragreserve gegen unvorhergesehene Einflüsse, bezieht sich nicht nur auf de Qualität der Bauwerksaussteifung. Vielmehr handelt es sich um Unempfindlichkeit, welche über Normen und technische Regeln hinausgeht, aber trotzdem zu beachten ist. Da verschiedene Lastpfade gleichzeitig auftreten können, informiert ein einzelner Zahlenwert über die Versagenswahrscheinlichkeit des schwächsten Traggliedes nur über die Wahrscheinlichkeit *eines* möglichen lokalen Versagens. Zusätzliche probabilistische Rechnungen sind durchzuführen, um andere mögliche Wege für die Lastabtragung zu ermitteln.

2.5 Risikoanalyse und Kostenminimierung

Zahlreiche Einflüsse können zu einer Reduzierung der Zuverlässigkeit von Bauwerken führen, welche sich durch Verluste der Tragfähigkeit oder Gebrauchstauglichkeit zeigen. Genauso wie Regeln für die Bemessung von Tragwerken geschaffen wurden, benötigt man auch für bestehende Bauwerke Entscheidungsgrundlagen zur Weiternutzung, zur möglichen Umnutzung bzw. zum Abriß. Art und Ort möglicher Schäden können sich je nach Lastbild oder -szenario unterschiedlich auswirken, weshalb deterministische Analysen allein nicht ausreichen. Da eine leichtsinnige Vorgehensweise zu Strukturversagen führen kann, übertriebene Vorsicht aber hohe Kosten verursacht, muß untersucht werden, wie man beides optimieren kann. Zuverlässigkeits- und Risikoanalysen bilden hierfür eine wichtige Grundlage, da sie als quantitatives Maß für eine Strukturbewertung dienen können.

Das Risiko für ein Bauwerksversagen hängt sowohl von der Eintrittsmöglichkeit und vom Ausmaß einer Schädigung ab, als auch von der Empfindlichkeit und Bedeutung des maßgebenden Bauteils. Unter Risikoanalyse versteht man demnach das Aufdecken von Schwachstellen und Fehlern, aber auch die Untersuchung der Tragwerkstopologie zur Ermittlung eventueller Folgen des Ausfall einzelner Tragglieder. Je höher der Sicherheitsanspruch ist, desto detaillierter muß die Risikoanalyse erfolgen, wobei fachfremden Personen alle wesentlichen Gesichtspunkte klar und verständlich darzulegen sind. Nur so können sie sinnvoll entscheiden. In diesem Zusammenhang kann man nicht oft genug betonen, daß das Versagensrisiko eines Bauwerkes zwar minimiert werden kann, sein Wert aber immer größer als Null bleiben wird. Ähnlich wie Sicherheitsbeiwerte im Rahmen einer Neubemessung festgelegt, müssen auch im Rahmen einer Zustandsbewertung Grenzwerte abhängig von Kosten durch Reparatur, Unterhaltung oder Ausfallzeiten, sowie von juristischen, politischen, ökologischen, denkmalpflegerischen und emotionalen Gesichtspunkten gewählt werden (Berz, ; 2000).

2.6 Beschreibung von Unsicherheiten

Kerngedanke beim Entwickeln der Fuzzy-Sets war für Zadeh der Wunsch, außer einer scharfen, Zugehörigkeit von Zahlenwerten zu einer Menge eine graduelle zuzulassen. Es ging ihm um die Beschreibung unscharfer Logik. Sein Ziel war es auch, Schlußfolgerungen zu ermöglichen, wenn nur Größenordnungen statt feste Größen vorliegen. Schon er erkannte die Tatsache, daß sich nur kalkulierbare Meßabweichungen (Unregelmäßigkeiten bei der Meßtechnik, Meßunsicherheit oder -unvollständigkeit) durch deterministische und probabilistische Verfahren adäquat berücksichtigen lassen. Jedoch schleichen sich bei der Modellbildung auch systematische Fehler durch falsche Formulierungen (menschlicher Irrtum) oder durch die Modellunsicherheit an sich ein. Jene könnten Strukturversagen verursachen, lassen sich allerdings durch mehr Simulationen oder bessere statistische Auswerteverfahren nicht reduzieren. Im Gegensatz zu den meisten mathematischen Algorithmen kann die Fuzzy-Logik auch nicht zufällige Fehler im Sinne von Unschärfe quantifizieren.

Gemäß ihrer ursprünglichen Bestimmung angewandt, hat sie sich z.B. bereits bestens in Konfliktsituationen, zur Entscheidungsfindung bei strategischen Spielen (z.B. Aktienauswahl), für Zuverlässigkeitsberechnungen, für Optimierungsaufgaben im Rahmen der Bauteilbemessung bewährt (Chao, R. und Ayub, B.; 1996, Möller, 1996 und Schnellenbach-Held, M.; 1996). Fuzzy-Sets verhindern, daß Informationen unrechtmäßig vereinfacht werden, da sie Unsicherheiten explizit quanitfizieren. Sie fassen ein Maß zwischen Unzuverlässigkeit und Zuverlässigkeit in eine mathematische Schreibweise und ergänzen somit die deterministische und probabilistische Zustandsbewertung in idealer Art und Weise. Anstatt unscharfe, genäherte oder vage Daten zwanghaft in scharfe Intervalle zu pressen und so eine nicht vorhandene Genauigkeit vorzutäuschen, beschreiben Fuzzy-Sets weiche Ergebnisse mit kontinuierlichen Intervallen der Form $M \in [0,1]$ statt $M = [0,1]$.

3 Ablauf der aktuellen Untersuchungsmethode

3.1 Auswahl von Schadensindizes und Sensoren zur Datenakquisition

Da viele Schäden nicht direkt erkennbar sind, kann die Schadenssimulation nur indirekt über die Strukturantwort erfolgen. Hierfür verwendet man spezifische Zustandsindikatoren[1] wie z.B. „Stützenauslenkung“, „Maximale Verformung“ oder „Energiedissipation“ in Verbindung mit mehr oder weniger aussagefähigen Modellen. Für die Datenakquisition werden Sensoren ausgewählt, die preiswert, leicht, gut zu installieren, sensitiv gegenüber der Schadensentwicklung und unempfindlich gegenüber Störungen sind. All diese Erfordernisse gleichzeitig zu befriedigen ist schwierig.

3.2 Datenanpassung und -analyse

Bei bestehenden Bauwerken stellen Unschärfe und Unvollständigkeit aufzunehmender Daten ein größeres Problem dar als bei Neubauten. Zusätzliche Informationen verbessern die Situation nur geringfügig. Im Ausgangsmodell innewohnende Unsicherheiten sind daher mit einem geeigneten Algorithmus zu beschreiben, ohne vorhandene Fehler zu vertuschen oder zu verstärken. Die „Methode der kleinsten Quadrate“ liefert selbst bei schwach fehlerhaften Eingangsdaten (unvermeidlicher Streubereich) oft falsche Ergebnisse. Um sowohl Meßfehler bei der Datenakquisition, als auch die Komplexität des angewandten Modells reduzieren zu können, sind die zu untersuchenden Daten zu analysieren und zu transformieren. Mit Hilfe von Kalibrierungs- und Regulierungsverfahren lassen sich Fehler zum Teil ausgleichen.

Obwohl die Dichte von Signalen erste Anhaltspunkte über den Strukturzustand liefert, zeigen sich wichtige Merkmale oft erst durch Selektion und Korrelation

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[1] Auswahl abhängig von der Beanspruchungsart (statisch, dynamisch, impulsartig)

der Daten. Um Signale später besser verarbeiten zu können und ausschließlich für die Interpretation erforderliche Informationen zu erhalten, werden sie zusätzlich skaliert, gefiltert oder synchronisiert. Durch Aufspalten von Erreger- und Antwortspektren in ihre nieder- und hochfrequenten Bestandteile lassen sich kleine Unregelmäßigkeiten infolge Rauschen, Wind, oder Zeitverzögerung im Zuge der Überlagerung ausgleichen. Trotz der Vorzüge zeitbezogener Schätzmethoden (Strukturantwort läßt sich in ihrer Dynamik vollständig erfassen, da der Zeitverlauf explizit berücksichtigt wird), wandelt man Impuls- meist in Frequenzfunktionen um, nachdem man die Zeit digital mit der Fast Fourier Transformation komprimiert hat.

3.3 Modellierung und Modellverbesserung

Bei der Modellbildung wird das Tragwerk idealisiert. Bei bestehenden Bauwerken ist dies nicht ganz so einfach wie bei einer Neubemessung. Das Modell muß den Anforderungen der Informationstechnik (Genauigkeit) und Anwender (Handhabbarkeit, Nutzerakzeptanz, Funktionalität, Zeitabhängigkeiten oder anderen Constraints) genügen, um das Bauwerk flexibel, fehlertolerant und weitgehend konsistent darzustellen zu können. Die durch Elimination von Widersprüchen und irreführenden Details verbundene Informationsreduktion auf ihre wesentlichen Bestandteile schafft Distanz zur Problemstellung und somit einen guten Überblick. Um bei der Bearbeitung unerwünschten, aber unvermeidlichen Verlust des Informationsgehaltes auszugleichen, sind verschiedene Datentypen (geometrisch, multimedial und verbal) miteinander kombiniert aufzunehmen und hinsichtlich ihrer unterschiedlichen Abstraktionsstufen bzw. Wertebereiche (Informationstiefe) darzustellen.

Da nicht nur Meßfehler (zufällig), sondern auch Modellfehler (nicht zufällig) möglich sind, werden Daten und Modelle verifiziert und validiert. Die Verifikation bezweckt ein vollständiges Modell mit, im mathematischen Sinn, konvergierenden Zielfunktionen, um Lösungsfehler zu vermeiden. Ein System wurde richtig modelliert, wenn die zur Modellbildung verwendeten Informationen rekonstruierbar sind. In der Forschung gilt eine Behauptung als bewährt oder „verifiziert“, wenn sie nicht im Rahmen einer Konfrontation mit der Wirklichkeit scheitert (Popper[2], K.R.; 1971). Die Validierung überprüft Detailinformationen auf ihre Korrektheit, um Fehler im Modell zu vermeiden. Dessen Struktur sollte innerhalb des betrachteten Bereichs mit der Systemstruktur übereinstimmen. Hat sich gezeigt, daß das richtige System modelliert wurde, zeigt die Evaluation am Ende, ob das Modell für den vorgeschriebenen Zweck passend und zuverlässig ist. Um vom Anwender akzeptiert zu werden, soll es so einfach wie möglich und so kompliziert wie nötig sein.

Im Rahmen der Systemidentifikation unterscheidet man Struktur- und Verhaltensmodelle. Während Strukturmodelle mit Hilfe der parametrischen probabilistischen Monte Carlo Simulation (auch sie erfaßt nicht alle wichtigen Daten, sondern nur eine Vielzahl möglicher Einflüsse), Baeysian Methode oder Latin Hypercube verbessert werden, geschieht dies bei Verhaltensmodellen durch nichtparametrische Regressionsanalysen oder Neuronale Netze. Letztere

[2] „Logik der Forschung“, 4. Auflage Tübingen

wiesen in der Vergangenheit Erfolge in unterschiedlichen Fachgebieten wie Maschinensteuerung, Qualitätskontrolle, Datenanalyse, Zustandserkennung und Klassenbildung auf. Deshalb entschlossen sich einige Experten, sie auch in der Bauwerksdiagnostik anzuwenden. Neuronale Netze optimieren Modellparameter anhand von bekannten Ein- und Ausgabepaaren. Obwohl sie gerade wegen ihrer Lernfähigkeit sehr beliebt sind, konnten sie bislang den komplexen Anforderungen an Zustandsanalysen nicht gerecht werden. Im Gegensatz zu parametrischen Verfahren können sie Strukturveränderungen oder Schäden weder differenzieren, noch deren Ursache aufzeigen. Selbst mit hohen Rechenaufwand ließen sich ihre Algorithmen nicht so formulieren, daß ihr Weg zum Untersuchungsergebnis nachvollziehbar wird. Nichtparametrisch arbeiten sie nämlich ohne räumlich oder physikalisch definiertes System (black-box). Selbst wenn sie mit qualitativ und quantitativ ausreichend Trainingspaaren gefüttert würden, verbessern sie bestenfalls „fuzzifizierte“ Lernregeln.

4 Fuzzy-Logik für eine bessere Zustandsbewertung

4.1 Einführung

Im Gegensatz zur Neubemessung eines Bauwerks oder zur konventionellen Parameteridentifikation (Ermittlung des Strukturverhaltens aufgrund einer Anregung), sind bei bestehenden Bauwerken Schädigungen zu diagnostizieren, ohne deren statisches System zu kennen. Bestenfalls lassen sich Konstruktionspläne oder Parameterangaben für die Zeit ihrer Errichtung beschaffen. Diese betreffen jedoch nur die Anfangssituation, wobei später eingetretene Zustände unberücksichtigt bleiben und somit auch Schädigungen. Zur Zeit werden deshalb für die Zustandsbewertung meist anhand von Photos subjektiv ausgewählte Schadensindizes benutzt, um im Rahmen der Systemidentifikation die Strukturtopologie ausgehend von Belastung und Bauwerksantwort zu ermitteln. Mit unscharfen Daten und oftmals inkonsistenten Zusatzinformationen können selbst die besten Verfahren zur Datenanalyse und Modellverbesserung maßgebende Parameter nur mit Unsicherheiten auswählen und auswerten.

Extreme Einwirkungsbedingungen, Bauwerksempfindlichkeit, nutzungsspezifische Anforderungen (Gefahrenpotential abhängig vom Stellenwert des Bauwerks) sowie Standort- und Umweltbedingungen (Bauwerk - Boden Interaktionen) müssen bei bestehenden Bauwerken gleichzeitig analysiert werden. Im Gegensatz zu Neubauten ist bei jenen allerdings das verfügbare statistische Datenmaterial unzureichend, weshalb sich der stochastische Charakter der Information schwer einschätzen läßt. Zuverlässigkeitsberechnungen wurden bisher nur für ausgewählte Tragsysteme (z.B. Atomkraftanlagen) und Schadensursachen (z.B. Erdbeben oder Impakt) durchgeführt. Auch Entscheidungsgrundlagen für akzeptable Risiken liegen nur zum Teil vor. Dennoch wurden bereits verschiedene Methoden und Verfahren zur Zuverlässigkeits- und Risikoanalyse erprobt, und diese insbesondere für Einzelelemente und Teilsysteme eingesetzt. Um allerdings das Tragsystem als Ganzes wirklichkeitsnah und objektiv betrachten zu können, ist auch die Beziehung der Einzelelemente zueinander adäquat zu berücksichtigen. Dies kann mit Hilfe von Verknüpfungsmodellen (Serien- bzw. Parallelschaltung) geschehen. Tragfähigkeit und Gebrauchsfähigkeit lassen sich nur sicherstellen, wenn nutzungstechnologische und bautechnische Funktionen in mehreren

Zuständen (bisherige Nutzung -Vergangenheit, gleiche Nutzung mit anderen Nutzungsparametern, geänderte Nutzung) bewertet werden. Neben deren Bauwerksempfindlichkeit sind alle relevanten Beanspruchungsparameter und Umweltbedingungen in Zusammenhang mit möglichen Schädigungsprozessen und Versagensvarianten zu betrachten. Sodann können die Einzelelemente mit dem mittleren Schadensausmaß η belegt werden, welche in die Rechnungen für die Versagensrisiko des Gesamtbauwerks eingehen.

Der Wissensmangel über Fehlermechanismen hat bisher ein systematisches, objektives Vorgehen verhindert, wodurch immer wieder wichtige Aspekte übersehen und unbedeutende überbewertet werden. Dieses Defizit hat man durch verfeinerte deterministische Nachweise auszugleichen versucht. Unschärfen in Geometrie, Material, Belastung, Randbedingungen und Modellbildung ist man mit probabilistischen Verfahren begegnet. Diese können allerdings nur zufällige Streuungen bei Lastannahmen oder Materialeigenschaften beherrschen. Deshalb haben deterministische und probabilistische Verfahren bisher die Schadensanalyse nur unmerklich verbessert. Aus Gründen der Sicherheit und Wirtschaftlichkeit fordern verantwortliche Bauherren deshalb eine Vorgehensweise, bei der wichtige Einzel- und Zwischenergebnisse nachvollziehbar sind.

Durch ihre besonderen Eigenschaften kann die Fuzzy-Logik bei zwei verschiedenen Problemstellungen bessere Ergebnisse liefern: Zum einen, wenn der reale Zusammenhang zwischen Schadensformen und -ursachen unklar ist, oder wenn die Modelldaten unvollständig bzw. das System für eine Modellierung mit mathematischen Formeln zu unübersichtlich ist (Modellunsicherheit). Zum anderen, wenn das Tragsystem zwar bekannt ist und beschrieben werden kann, maßgebliche Parameter aber mit Unsicherheiten behaftet sind (Datenunsicherheit). Im Rahmen einer Bauteileinstufung in Qualitätsklassen mit Fuzzy-Zugehörigkeitsfunktionen stellen informelle und linguistische Unsicherheiten kein Problem mehr dar. Bei der Untersuchung von Merkmalen innerhalb der Datenanalyse bewältigt die Fuzzy-Logik höherdimensionale Suchräume, welche zu groß wären, um vom menschlichen Beobachter auf ein Mal überblickt werden zu können (für z.B. drei Ursachen und zwei Wirkungen wäre eine derartige Rechnung nicht erforderlich, da hier die Zuordnung eindeutig festgelegt werden könnte).

Obwohl jedes Bauwerk einzigartig ist und sich die Realität nie vollständig beschreiben läßt, ist für Tragwerke mit ähnlichen Eigenschaften die gleiche Vorgehensweise zu empfehlen, deren zeitlicher Ablauf in Abbildung 5, 7 und 8 von Abschnitt 4.2 deutlich wird:

Nachdem ein akzeptiertes Risikoniveau festgelegt worden ist, beschreibt man mit Hilfe eines vorläufigen Modells die Geometrie aller wesentlichen Tragglieder sowie die Systemkonfiguration. Gleiches gilt für die horizontale und vertikale Lastabtragung, Lastgeschichte, Materialparameter und Randbedingungen. Man definiert lokalen (ein Tragelement) und globalen (Gesamtsystem) Schaden. Dann identifiziert man mögliche Schwachstellen und legt sinnvolle Grenzwerte für wahrscheinliche und sensitive Zustandsparameter auf Material- und Strukturebene fest.

Abbildung 2 erläutert wesentliche Bestandteile einer Zustandsanalyse.

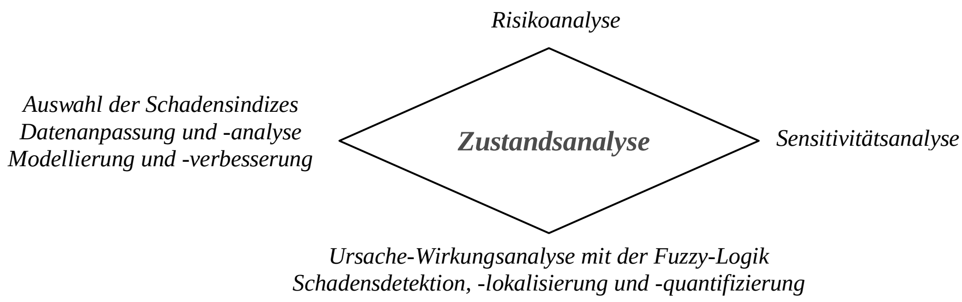


Abbildung 2: Bestandteile einer ganzheitlichen Zustandsanalyse

Um ein Bauwerk in Qualitätsklassen nach Abschnitt 4.2.1 einstufen zu können, sind je nach Bedarf eine Bewertungsstufe (nur „Grobeinstufung“ nach 4.2.2), zwei Bewertungsstufen (4.2.2 und „Abschätzende Beurteilung“ nach 4.2.3) oder drei Bewertungsstufen (4.2.2, 4.2.3 und „Weitergehende Untersuchungen“ nach 4.2.4 unter Anwendung von Fuzzy-Algorithmen nach 4.4) erforderlich. Das Risiko für das Versagen eines Bauteils oder des gesamten Bauwerks kann im Anschluß daran nach Abschnitt 4.3. ermittelt werden.

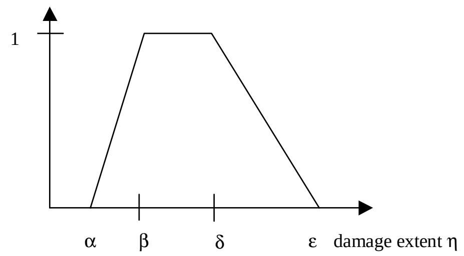
4.2 Vorschlag für ein ganzheitliches Bewertungsverfahren

Aufgrund der beschränkten finanziellen Mittel können nicht alle geschädigten Bauwerke mit gleicher Genauigkeit beurteilt werden. Es wird deshalb ein dreistufiges Verfahren vorgeschlagen, das eine Einstufung in Qualitätsklassen ermöglichen soll. Die „Grobeinstufung“ basiert zumeist auf visueller Inaugenschiennahme und einfachen Vergleichsrechnungen. Gelingt so die Einstufung nicht, wird eine „Abschätzende Beurteilung“ vorgenommen. Sie benutzt einen Fehlerbaum zur Beschreibung möglicher Fehlermechanismen, um auf die maßgebliche Versagensart schließen zu können. In Ausnahmefällen werden im Rahmen von „Weitergehenden Untersuchungen“ umfangreiche Rechnungen auf der Basis von Fuzzy-Algorithmen durchgeführt.

4.2.1 Fuzzy-Zugehörigkeitsfunktionen zur Einstufung in Qualitätsklassen

Trapezförmige Fuzzy - Zugehörigkeitsfunktionen können zur Einstufung der Tragglieder E_i in Qualitätsklassen gemäß Abbildung 3 und 4 angewandt werden (Tillmann, T.; 1996):

$$A = [x_i ; \mu_A(x) = (\alpha, \beta, \delta, \varepsilon)] \textit{ with } 0 < \mu_A(x) \leq 1 .$$



$$\mu(DE_i) = \begin{cases} 0, & x < \alpha \\ \dfrac{x-\alpha}{\beta-\alpha}, & \alpha \leq x \leq \beta \\ 1, & \beta \leq x \leq \delta \\ \dfrac{\varepsilon - x}{\varepsilon - \delta} & \delta \leq x \leq \varepsilon \\ 0, & x > \varepsilon \end{cases}$$

Abbildung 3: Mathematische Beschreibung von Trapezförmigen Zugehörigkeitsfunktionen

Eine Zugehörigkeitsfunktion $\mu(DE_i)$ kann die Klassen „sehr gut, gut, mäßig, mangelhaft, abbruchreif“ mit „none (N), slight (SL), small (SM), moderate (MD), considerable (CN), severe (SV), very severe (VS), destructive (DS)“ festschreiben (Jendo, S. et altri; 1998). So bedeutet die Referenz-Zustandsmatrix S (DE_1, DE_2, DE_3, DE_4) = (SV, N, MD, MD) z.B., daß S für DE_1 mit „severe“, DE_2 mit „none“, DE_3 und DE_4 mit „moderate“ eingestuft werden kann.

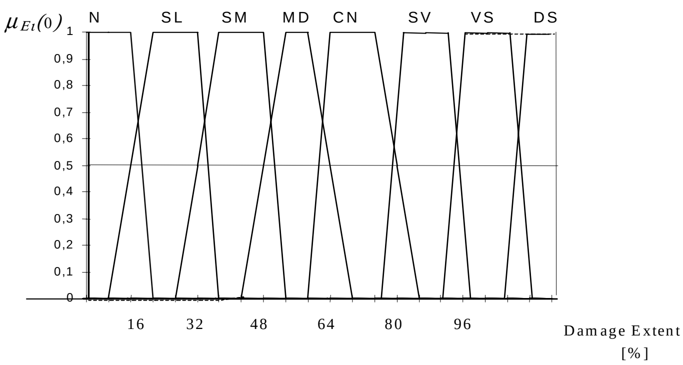


Abbildung 4: Zugehörigkeitsfunktionen DE_i

Ein Schadensausmaß von 80 % mit einem Zugehörigkeitsgrad von $\mu(x) = 0.6$ gehört z.B. zur Kategorie „considerable“ und mit einem Zugehörigkeitsgrad von $\mu(x) = 0.6$ gleichzeitig auch zu „severe“. Ein und dasselbe Schadensausmaß kann also gleichzeitig zu zwei verschiedenen $\mu(x)$ gehören.

4.2.2 Grobeinstufung

Die Grobeinstufung dient dazu, lebensrettende Sofortmaßnahmen einzuleiten und die Bauteile in Qualitätsklassen nach Abschnitt 4.2.1 einzustufen. So kann man einen Großteil der Bauwerke von einer abschätzenden Beurteilung ausschließen. Abbildung 5 zeigt ein Flußdiagramm, wie sie erfolgen kann:

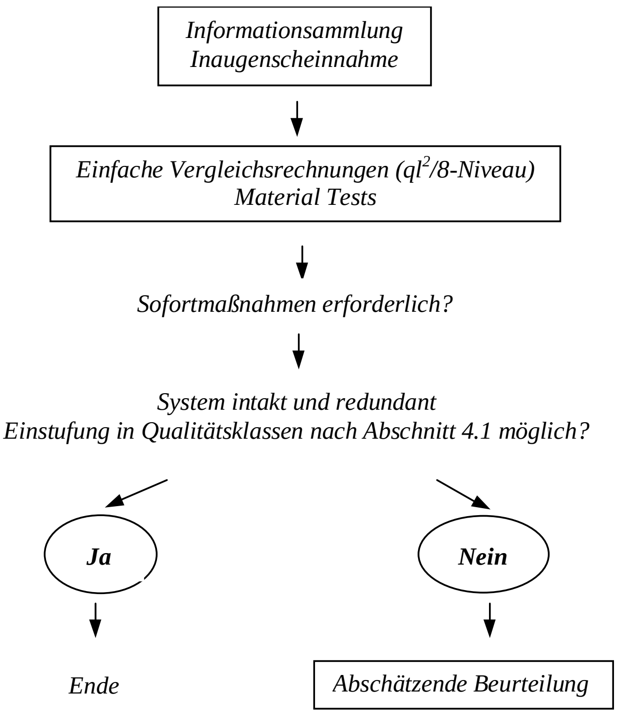


Abbildung 5: Flußdiagramm für eine Grobeinstufung

4.2.3 Abschätzende Beurteilung

Nur im Bedarfsfall, d.h. wenn nach der Erstanalyse keine Entscheidung getroffen werden kann, folgt die „Abschätzende Beurteilung“ mit ausführlicher Geometrieaufnahme und Materialerkundung. Detailphotos von kritischen Bereichen bzw. Weitwinkelaufnahmen des Gesamtbereichs sollen über Tragglieder und Systemtoplogie informieren.

Mit Hilfe eines Fehlerbaumes lassen sich Fehlermechanismen und -pfade systematisch untersuchen. Von einer begrenzten Zahl an Schadensindizes[3] auszugehen ist nicht immer ausreichend, da ein einzelner Defekt nicht zwingend auf einen bestimmten Schaden bzw. eine eindeutige Versagensart hinweist (kann aber zusammen mit einem unvorhergesehenem Ereignis maßgebend sein).

[3] Auswahl entsprechend letzten Abschnitt dieses Kapitels, sowie nach Kapitel 3.1

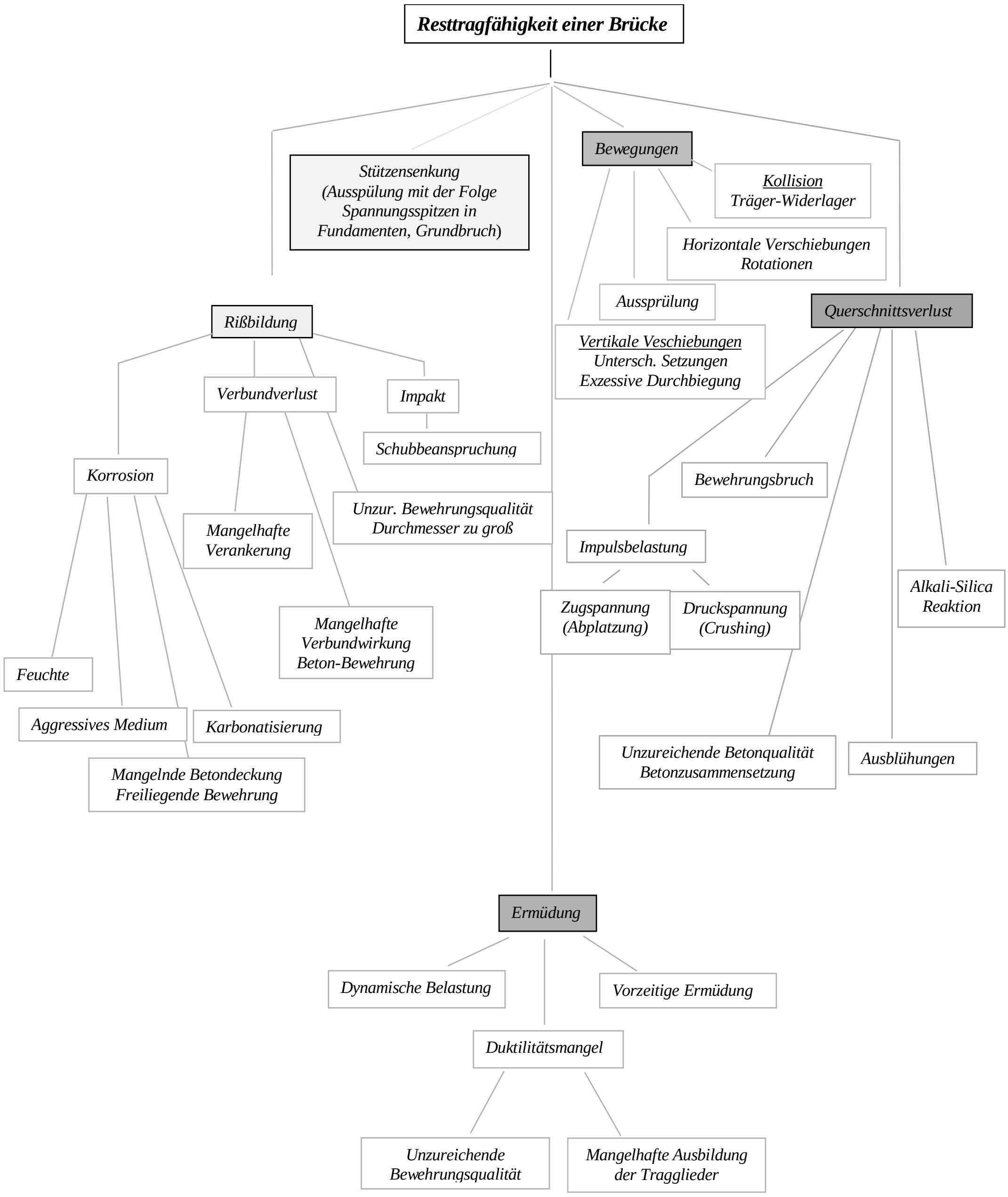


Abbildung 6: Beispiel für einen Fehlerbaum im Rahmen einer Brückeneinstufung

Ein Fehlerbaum wird von oben nach unten aufgebaut, solange bis alle qualitativen Schlußfolgerungen feststehen.

Umweltbedingte Einwirkungen	*Stabilität / Festigkeit*	*Menschliches Versagen*
Erdbeben	*Materialversagen*	*Wissensmangel*
Wind	*Versagen im Bauzustand*	*Ungenügende Bemessung*
Flut (Wasserdruck)	*Kollision / Impakt*	*Mangelhafte Kooperation*
Feuer	*Ermüdung*	*Fehlerhafte Bauausführung*
Explosion	*Progrtessives Versagen*	*Mangelhafte Sicherheitsvorkehrungen*

Tabelle 1: Schadensereignisse und ihre Besonderheiten

Verschiedene Schadensereignisse unterschiedlicher Art und Herkunft bewirken vielfältige Bauwerksreaktionen (siehe Tabelle 1). So weichen z.B. durch Feuer bzw. Hurrikan verursachte Schäden, in wichtigen Aspekten voneinander ab. Statische Beanspruchung erzeugt Bauteilverdrehung, dynamische Beanspruchung Energiedissipation, und Impulsbelastung größere lokale Verformungen. Entsprechend sind hier also die Schadensindizes „Rotationsduktilität“ oder „Krümmungsduktilität“, „Energiedissipation“ und maximale Verformung jeweils maßgebend. In jedem Fall ist zu prüfen, ob die erforderliche Tragfähigkeit auch nach dem Ausfall einzelner Tragglieder gewährleistet ist. Eine Vergleichsrechnung verdeutlicht dies an Submodellen, welche gleichzeitig als Datenintegrationsebene zur Unterstützung der einzelnen Experten dienen. Gedanklich werden hier schrittweise einzelne Bauteile entfernt, um zu zeigen welche Rolle jene im Gesamtsystem einnehmen (*wo* wird ein Bauteil *wie* beansprucht?) Sodann kann eine Einstufung der Bauteile in Qualitätsklassen nach Abschnitt 4.2.1 erfolgen. Abbildung 7 zeigt ein Abflußdiagramm für eine „Abschätzende Beurteilung“:

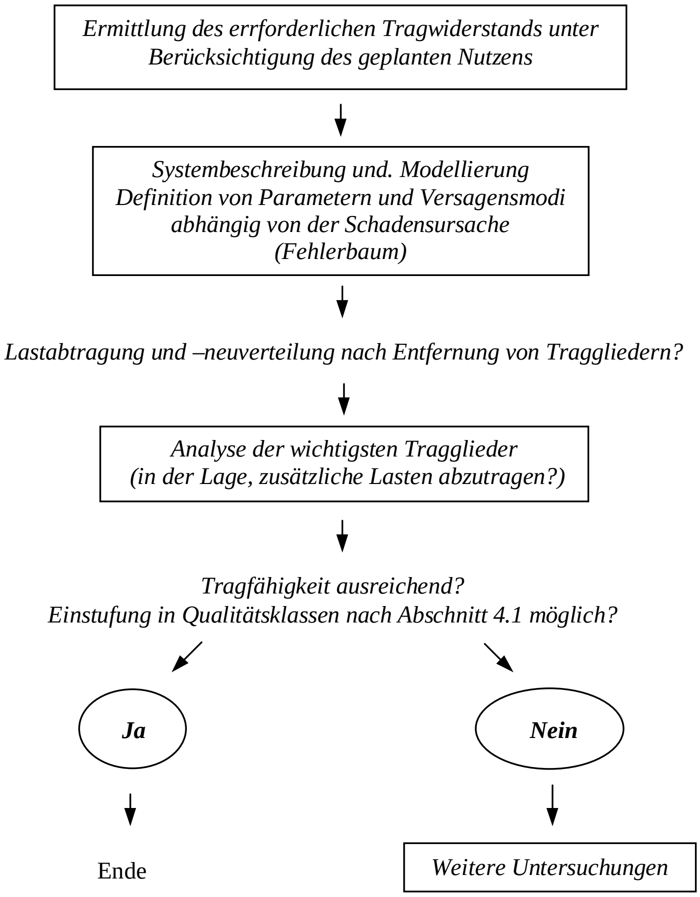


Abbildung 7: Flußdiagramm für eine Abschätzende Beurteilung

4.2.4 Weitergehende Untersuchungen

Besondere Umstände können weitergehende Untersuchungen zur Ermittlung bislang unberücksichtigter, zusätzlicher Tragreserven erfordern. Die digitale Photogrammetrie oder Tachymetrie und Laserscanner veranschaulichen beispielsweise die räumliche Tragstruktur, erfassen sogar Verschiebungen, Setzungen und Rißgeometrie. Da sie auf Art und Ausmaß einer Schädigung hinweisen. Somit lassen sich mögliche Risiken, erforderliche Ressourcen und entstehende Kosten ermitteln. Derartige Fakten können dem fachfremden Personenkreis als Grundlage für Entscheidungen dienen. In diesem Untersuchungsstadium sollte man Unschärfen in bezug auf Struktur-, Last- oder Materialparameter, sowie Fehler bei der Modellierung nicht mehr vernachlässigen. Mit Hilfe der Fuzzy-Logik gemäß Abschnitt 4.4 läßt sich sowohl der maßgebliche Versagensmodus, als auch das Schadensausmaß insgesamt bestimmen. Eine Einstufung in Qualitätsklassen nach Abbildung 3 und 4 kann somit erfolgen. Abbildung 8 zeigt ein Flußdiagramm für weitergehende Untersuchungen:

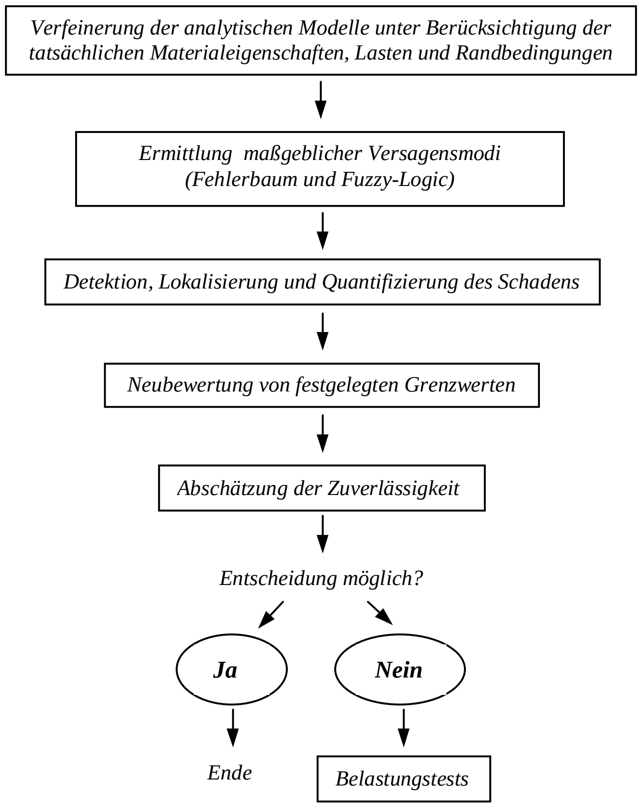


Abbildung 8: Flußdiagramm für Weitergehende Untersuchungen

4.3 Ermittlung des Risikos

Das mittlere Schadensausmaß η, der Prognosefaktor L_i gemäß Tabelle 2 und die Bauteilbedeutung I gemäß Tabelle 3 bestimmen maßgeblich das Risiko $p_i = \eta^I L$ eines Ereignisses (kleiner gleich 1). $L_{top} = L_1 + L_2 + L_3 ... \geq 1$ ist nicht zu verwechseln mit p_i, da er die Richtigkeit einer Beurteilung prognostiziert (trifft für mehrere Schadensformen zu) und deshalb den Wert 1 übersteigen kann.

$L = 1$	*Versagen ist sicher*
0.9	*Sehr wahrscheinlich*
0.7	*Wahrscheinlich*
0.55	*Ziemlich wahrscheinlich*
0.35	*Ziemlich unwahrscheinlich*
0.1	*Unwahrscheinlich*
0	*Keinesfalls*

Tabelle 2: Prognosefaktor für das Auftreten eines Versagens

I=1	*Entscheidend für Sicherheit*
I=0.9	*Entscheidend für Gebrauchsfähigkeit*
I=0.8	*Versagen einzelner Tragglieder*
I=0.6	*Unbedeutend für die Sicherheit*

Tabelle 3: Bedeutung des Tragglieds für das Gesamtsystem

Genauso wie verschiedene Versagensmodi eine Reihe von sich gegenseitig beeinflussenden Ursachen haben können, hängt auch das Risiko für ein Versagen des Systems nicht nur von Versagensrisiken einzelner Tragglieder ab, sondern auch von deren Anordnung. Da verschiedene Last- bzw. Verformungspfade gleichzeitig existieren können, reicht es nicht aus, das momentan schwächste Glied in deterministischem Sinne zu betrachten. Alle anderen Systemkomponenten könnten zum schwächsten Glied in der „Kette“ werden, weshalb z.B. große Systeme besonders anfällig für ein globales Versagen sind. Je duktiler die Komponenten sind, desto besser kann ihre Funktion durch Reihenkoppelung der Einzelrisiken idealisiert werden. Für reihen[4]geschaltete statisch bestimmte Systeme (Korrelationskoffizient > 0) entspricht das Risiko für die gesamte Struktur (Kraemer, U.; 1980, Fleischer, D.; 1988,Ching-Hsue, C. et altri; 1993 und Rackwitz; 1997):

$$\max_{i=1}^{n} p_i < P_f < 1 - \prod_{i=1}^{n} (1 - p_i)$$

$$P_f = P(g_1 \leq 0 \cup g_2 \leq 0 \cup .....g_n \leq 0).$$

Für parallel[5]geschaltete statisch unbestimmte Systeme (Korrelationskoffizient > 0) gilt (Kraemer, U.; 1980 und Rosowski, D.V.; 1997):

$$\prod_{i=1}^{n} p_i < P_f < \min_{i=1}^{n} p_i$$

$$P_f = P(g_1 \leq 0 \cap g_2 \leq 0 \cap .....g_n \leq 0)$$

Das gerechnete Versagensrisiko für die Gesamtstruktur wird am Ende mit dem zuvor festgelegten „Zielrisiko“ verglichen. Hierbei unterscheidet man ähnlich wie bei der Versagenswahrscheinlichkeit eine Gebrauchsgrenze mit $\leq 10^{-4}$ und eine Sicherheitsgrenze mit $\leq 10^{-6}$. Obwohl für „Low Probability-High Damage Events“ statistisch verwertbare Daten kaum verfügbar sind, kann ein akzeptierbares Risiko abhängig von einer zuvor definierten Versagenswahrscheinlichkeit mit $< 10^{-5}$ pro Jahr und pro Person festgelegt werden (Ellingwood, B.R.; 1994). Am besten vergleicht man bekannte Risiken für übliche Beschäftigungen indirekt über deren Eintretenswahrscheinlichkeit (diejenige für ein Zugunglück liegt bei 10^{-6} und für Tod aufgrund eines

[4] Versagen eines Tragglieds bedeutet Systemversagen
[5] Versagen eines Tragglieds führt nicht unbedingt zu Systemversagen

Blitzschlages bei 5×10^{-6}), ein hohes mittlere Schadensausmaß mit einer geringen Prognosefaktor zählt schwerwiegender, als umgekehrt (Ammann, W., 2000 und Berz, G.; 2000). So gilt z.B. der Tod von zehn Menschen bei zehn Unfällen durch private Autos als weniger schlimm als der Verlust von zehn Menschenleben bei einer einzigen Katastrophe. Es ist einfacher, ein Gebäude sicher zu bemessen, als sein „Überlebens"-Risiko an sich zu bewerten. Für letzteres benötigt man Daten über geschädigte Bauwerke. Da jene nicht uneingeschränkt verfügbar sind, ist keine Aussage in Form eines Zahlenwertes möglich. Für eine Entscheidung bezüglich Abriß oder Reparatur reicht jedoch eine Information in Form von Bereichswerten aus, weshalb de Fuzzy-Logik als Werkzeug dienen kann. Mit Hilfe von trapezförmigen Fuzzy-Zugehörigkeitsfunktionen (siehe Abschnitt 4.2.1) lassen sich sowohl Bauteile, als auch das gesamte Tragwerk in Qualitätsklassen einstufen. Mit derart unscharfen Daten würden klassische Verfahren kaum vergleichbare Aussagen zulassen, sondern nur eine Entscheidung für „ja" oder „nein" bzw. „gut" oder „schlecht".

4.4 Singletons zur Beschreibung von Ursache-Wirkungsbeziehungen

Reichen die vorhandenen Erkenntnisse nicht aus, um Schadensart und -maß eindeutig festzulegen, z.B. wenn die Beziehung zwischen Ursache und Wirkung von Schäden unklar ist, wird eine Rechnung basierend auf Fuzzy-Singletons eingesetzt. Obwohl Singletons die Mikrostruktur nicht so detailliert beschreiben wie etwa trapezformige Fuzzy-Funktionen, ermöglichen sie einen Vergleich und zeigen zumindest dominante Versagensmechanismen sowie deren Beziehung an. Zahlreiche Untersuchungen haben gezeigt, daß die Form von Zugehörigkeitsfunktionen (singulär, dreiecksförmig, trapezförmig, parabolisch etc.) nur einen geringen Einfluß auf das Ergebnis hat. Fuzzy-Matrizen aus Singletons spezifizieren, wie stark Schadensursache V_i und –wirkung q_i verknüpft sind. Die Fuzzy-Vektoren K geben an, wie stark man mit einer bestimmten Schadensform rechnen muss und dienen als Korrekturfaktoren (Chao, C.J. et altri; 1998, Jendo, S. et altri; 1998, Pandey, P. C.; 1995 and Ching, C.J. et altri; 1998). Während der Beziehungsraum R aus den Singletons μ_{ij} besteht, setzt sich K aus den Singletons k_i zusammen. Die Ursachevektoren ergeben sich so mit

$$V_1 = (\mu_{11} / \mu_{21} / \mu_{31} \ldots\ldots\ldots\ldots \mu_{m1})$$
$$V_2 = (\mu_{12} / \mu_{22} / \mu_{32} \ldots\ldots\ldots\ldots \mu_{m2})$$
$$.$$
$$.$$
$$V_n = (\mu_{1n} / \mu_{2n} / \mu_{3n} \ldots\ldots\ldots\ldots \mu_{mn})$$

$$R = \begin{array}{c} \\ q_1 \\ q_2 \\ . \\ q_m \end{array} \begin{array}{c} \begin{array}{cccc} V_1 & & ..... & V_n \end{array} \\ \begin{bmatrix} \mu_{11} & . & . & \mu_{1n} \\ \mu_{21} & . & . & \mu_{2n} \\ . & . & . & . \\ \mu_{m1} & . & . & \mu_{mn} \end{bmatrix} \end{array}$$

$$K = (k_1, k_2 \ldots\ldots k_r).$$

Die relative Bedeutung von Schadensmerkmalen für ein Bauteil wie „Rißbreite ist wichtiger als Rißlänge und als der Abstand zum nächsten Knoten“ oder „Scherbeanspruchung ist gefährlicher als Biegung“ wird beschrieben durch den „Importance Factor“ *H*:

$$H = \begin{array}{c} \\ q_1 \\ q_2 \\ . \\ q_m \end{array} \begin{array}{c} \begin{array}{cccc} q_1 & & ..... & q_m \end{array} \\ \begin{bmatrix} h_{11} & . & . & h_{1m} \\ h_{21} & . & . & h_{2m} \\ . & . & . & . \\ h_{m1} & . & . & h_{mm} \end{bmatrix} \end{array}$$

h_{ij} bedeutet, daß q_i um Faktor h_{ij} bedeutender ist, als q_j.

Mit $\sum_{i=1}^{m} w^*_i = 1$ und $w^* > 0$ gilt für den “Weighting Vector” W^*

$$W^* = \left(\sum_{j=1}^{m} h_{1j}, \sum_{j=1}^{m} h_{2j}, \ldots, \sum_{j=1}^{m} h_{mj}, \right) = (w^*_1, w^*_2, \ldots w^*_m).$$

Es geht nun darum, herauszufinden, welche der beiden Schadensursachen V_i oder V_j eher maßgeblich ist in bezug auf eine bestimmte Schadenswirkung. Die Ursachen V_i und V_j eines jeden Fuzzy-Paares werden verglichen mit Hilfe des “Weighted Hamming Distance” d_w

$$d_w(V_i, V_j) = \sum_{k=1}^{m} w^* [\mu_{ik} - \mu_{jk}].$$

Für $d_w(V_i, V_j) > 0$ ist V_i maßgebend, für $d_w(V_i, V_j) < 0$ ist V_j und für $d_w(V_i, V_j) = 0$ sowohl V_i, als auch V_j. Jeder Rechenschritt schließt eine Ursache aus und legt die andere als maßgebliche fest. Der Grad, mit welchem eine bestimmte Ursache für einen Schaden verantwortlich ist, wird ermittelt mit dem "Confirmation degree" C_i

$$C_i = \begin{pmatrix} w^*_1 \times k_1 \\ w^*_2 \times k_2 \\ \vdots \quad \vdots \\ w_n \times k_n \end{pmatrix} \times \begin{pmatrix} V_{i1} \\ V_{i2} \\ \vdots \\ C_{in} \end{pmatrix}$$

$$C_i = \sum_{j=1}^{s} (W^* \times K) \times V_i, \ i = 1, 2, ..., n.$$

Abhängig von C_i werden im Rahmen eines Defuzzifizierungsprozesses Schadensindizes D_i festgelegt. Je nach ihrer Bedeutung oder ihres lokalen Einflusses auf die Elementtragfähigkeit bzw. auf die Systemtragfähgikeit belegt man sie mit Wichtungsfaktoren w_i und errechnet am Ende einen mittleren Schadensindex $D_w = \frac{D_i^2 w_i}{\sum D_i^2}$. Sollen Art, Bedeutung und Zuverlässigkeit oder Glaubwürdigkeit der erhaltenen Information berücksichtigt werden, sind zusätzliche Koeffizienten γ gemäß Tabelle 4 festzulegen, mit denen die einzelnen Schadensindizes D_i dividiert werden: daraus ergibt sich das mittlere Schadensausmaß mit $\eta = \frac{(D_i / \gamma)^2 w_i}{\sum D^2{}_i}$.

Art und Bedeutung der Information		*Zuverlässigkeit der Information* γ	
		Mäßig	*Gut*
Vergleichsrechnungen überprüft	*Teilweise*	*0.3*	*0.4*
	Vollständig	*0.8*	*1.0*
Materialtests durchgeführt	*Teilweise*	*0.3*	*0.4*
	Im Detail	*0.8*	*1.0*
Vorwarnung	*Keine Risse oder Verformungen*	*0.4*	*0.6*
	Risse oder Verformungen	*0.6*	*0.8*
	Risse und Verformungen	*1.0*	*1.0.*
Einfluß auf nichttragende Elemente	*Vernachlässigbar*	*1.0*	*1.0*
	Bedeutend	*0.5*	*0.7*

Tabelle 4: Koeffizienten zur Berücksichtigung von Informationsart, -bedeutung und -zuverlässigkeit

5 Erläuterung der Methodik durch Beispiele

Da nicht alle Versagensmechanismen eindeutig zu erkennen sind, erläutern zwei Beispiele die Anwendung des Fuzzy-Algorithmus und des gesamten Bewertungskonzeptes, wobei die nötigen Rechnungen in der englischen Hauptveröffentlichung beschrieben sind. Um die Robustheit des Verfahrens zu belegen, wird geprüft, ob das Ergebnis auch bei leichter Abänderung einzelner Zahlenwerte aufgrund verschiedener Expertenmeinungen logisch bleibt.

6 Zusammenfassung

Während die Bemessung von Tragwerken im allgemeinen durch Vorschriften geregelt ist, gibt es für die Zustandsbewertung bestehender Bauwerken noch keine objektiven Richtlinien. Viele Experten sind mit der neuen Problematik (Systemidentifikation anhand von Belastung und daraus entstehender Strukturantwort) noch nicht vertraut und begnügen sich daher mit Kompromißlösungen. Für viele Bauherren ist dies unbefriedigend, weshalb in dieser Arbeit eine objektivere und wirklichkeitsnähere Zustandsbewertung vorgestellt wird.

Über die hierfür erforderlich sind theoretische Grundlagen der Schadensanalyse, Methoden und Techniken zur Geometrie- und Materialerkundung, Duktilität und Energieabsorption, Risikoanalyse und Beschreibung von Unsicherheiten handelt es sich in Kapitel 2.1 bis 2.6 werden zu Beginn erläutert. Da nicht alle Schäden offensichtlich sind, kombiniert man zur Zeit mehrere Zustandsindikatoren, bereitet die registrierten Daten gezielt auf, und integriert sie vor einer endgültigen Bewertung in ein validiertes Modell (Kapitel 3). Werden deterministische Nachweismethoden mit probabilstischen kombiniert, lassen sich nur zufällige Fehler problemlos minimieren. Systematische Fehler durch ungenaue Modellierung oder vagem Wissen bleiben jedoch bestehen. Daß Entscheidungsträger mit unsicheren, oft sogar widersprüchlichen Angaben subjektiv urteilen, ist also nicht zu vermeiden.

In dieser Arbeit wird gezeigt, wie mit Hilfe eines dreistufigen Bewertungsverfahrens Tragglieder in Qualitätsklassen eingestuft werden können (Kapitel 4.2). Abhängig von ihrem mittleren Schadensausmaß η, ihrer Strukturbedeutung *I* (wiederum von ihrem Stellenwert bzw. den Konsequenzen ihrer Schädigung abhängig) und ihrem Prognosefaktor *L* ergibt sich ihr Versagensrisiko mit $p_i = \eta^I L$. Das Risiko für eine Versagen der Gesamtstruktur wird aus der Topologie ermittelt (Kapitel 4.3).

Wenn das mittlere Schadensausmaß nicht eindeutig festgelegt werden kann, oder wenn die Material-, Geometrie- oder Lastangaben vage sind, wird im Rahmen „Weitergehender Untersuchungen“ ein mathematisches Verfahren basierend auf der Fuzzy-Logik vorgeschlagen. Es filtert auch bei komplexen Ursache-Wirkungsbeziehungen die dominierende Schadensursache heraus und vermeidet, daß mit Unsicherheiten behaftete Parameter für zuverlässige Absolutwerte gehalten werden. Um den mittleren Schadensindex und daraus das Risiko zu berechnen, werden die einzelnen Schadensindizes D_i (je nach Fehlermodus) abhängig von ihrer Bedeutung mit Wichtungsfaktoren w_i belegt, und zusätzlich je nach Art, Bedeutung und Zuverlässigkeit der erhaltenen Information durch γ dividiert (Kapitel 4.4).

Die mit der vorgestellten Methode erhaltenen Ergebnisse sind in sich schlüssig und für verschiedene Tragsysteme gültig. Kapitel 4 beschreibt ein neues Verfahren zur Analyse komplexer Versagensmechanismen und ermöglicht *nachvollziehbare* Schlußfolgerungen. Obwohl die damit verbundenen Lösungsmöglichkeiten vielversprechend scheinen, legen noch nicht zu behebende Schwierigkeiten im Bereich der Fuzzifizierung und Defuzzifizierung fortdauernde Forschungsaktivitäten nahe. Zukünftige Entwicklungsmöglichkeiten weisen auf folgendes: Parametereigenschaften könnten durch Merkmalsauswahl und -extraktion mit eigens hierfür entwickelten Verfahren besser beschrieben werden. Zusätzliche Fuzzy-Algorithmen können Beziehungsräume abhängig von durch Experten festgelegten Einzelwerten, veranschaulichen. Drittens könnten Fuzzy-Logik und Neruonale Netze zusammen die Einsatzmöglickeiten im Bereich der Schadensdiagnostik wesentlich erweitern und die Leistungsfähigkeit von Rechenverfahren weiter erhöhen.