Failure consequences and reliability acceptance criteria for exceptional building structures
A study taking basis in the failure of the World Trade Center Twin Towers

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Failure Consequences and Reliability Acceptance Criteria for Exceptional Building Structures

A Study taking Basis in the Failure of the World Trade Center Twin Towers

Michael H. Faber, Oliver Kübler
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Institute of Structural Engineering
Swiss Federal Institute of Technology

Zurich
July 2004
Preface

The present research project was performed in the period April 2002-March 2004 and was funded by the Swiss National Science Foundation under grant number 2100-066770.

During the project period a number of publications and presentations at conferences on the topics of the project have resulted in many interesting discussions with several researchers. These discussions have provided a valuable input to the project. Specifically the project team warmly thanks Prof. Dr.-Ing. habil. R. Rackwitz, Technical University of Munich, Germany, for valuable discussions and input concerning aspects of optimality and affordable societal life saving costs. Prof. Dr. M. Haller, University St. Gallen, Switzerland is greatly acknowledged for discussions and viewpoints in regard to the socio-economical impact of hazards on the society. Moreover, Yngve Abrahamsen from the Swiss Institute for Business Cycle Research, Zurich, Switzerland, contributed with helpful discussions on the topic of macroeconomics. Finally Prof. Dr. A. Dazio and research assistant D. Buzzini, Swiss Federal Institute of Technology, Zurich, Switzerland, are sincerely thanked for their investigations regarding robustness of structures against progressive collapse.

Zurich, July 2004

Michael Havbro Faber
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1 Introduction

1.1 Background, motivation and scope of the research project

1.1.1 Background and motivation

The modern codes for the design of structures have been calibrated for normal structures, i.e. such that structures for normal purposes built in usual dimensions and designs, using well-known materials and constructed and maintained according to established procedures achieve an adequate and homogeneous level of reliability – in a cost efficient manner. Structures, which are not normal in the above-mentioned sense fall beyond the application area of the design codes and may be seen as exceptional structures. One particular type of exceptional structure is building structures with a high concentration of business enterprises and not least people and know-how. Failure of such structures may have enormous consequences not only for the owners of the buildings, the people and enterprises occupying the buildings but also for society in general. To establish a rational design basis for this type of buildings is a task involving the consideration of several aspects similar to the case when design codes for the design of ordinary structures is considered. However, significant differences arise in connection with the assessment of the optimal level of safety for such structures. Due to the potential enormous consequences in case of failure, the optimal level of safety to be applied as basis for the design may differ significantly from the optimal level of safety for so-called ordinary structures. This in turn may imply that when designing such structures, other load cases, failure modes and hazard scenarios need to be considered and new principles for the design may be relevant.

1.1.2 Scope of the research project

Despite the atypical and senseless cause of the tragic failure of the World Trade Centre Twin Towers in New York on September 11, 2001 it might be possible to learn something from the event whereby the design basis of similar future structures could be enhanced. Of special importance is the assessment of the actual consequences in case of failure. The realistic assessment of these is a prerequisite for establishing an optimal level of safety of such structures, i.e. a cornerstone in the design basis for such structures. The scope of the present project is to analyze the event of the failure of the World Trade Centre towers in regard to the consequences of the structural failure - not including the consequences attributable to the cause of the event (malevolence). The evaluations include an assessment of cost consequences on a societal level taking into account adverse effects due to the loss of lives, loss of businesses and material losses. Finally based on the findings in regard to consequences, initial principal studies are performed for the assessment of optimal safety strategies for such structures.

1.2 Executive summary and conclusions

1.2.1 Mapping, assessment and modeling of consequences

The present research report is initially concerned about the mapping of the consequences following the incident of the failure of the World Trade Center Twin Towers on September 11, 2001.

A thorough reporting and assessment is given on the reported number of fatalities, economical losses as well as potential impacts to the environment and cultural assets. The main focus is, however,
devoted to the assessment and differentiation of economical losses. The types of consequences considered and their differentiation is illustrated in Figure 1.1.

![Figure 1.1](image)

**Figure 1.1** Different types of consequences mapped and assessed within the present report.

**Table 1.1** Summary of assessed consequences following the failure of the WTC Twin Towers.

<table>
<thead>
<tr>
<th>Consequence type</th>
<th>Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
</tr>
<tr>
<td>Rescue &amp; clean-up</td>
<td>1.7</td>
</tr>
<tr>
<td>Property</td>
<td>19.2</td>
</tr>
<tr>
<td>WTC Twin Towers</td>
<td>4.7</td>
</tr>
<tr>
<td>Other destroyed buildings</td>
<td>2.0</td>
</tr>
<tr>
<td>Damaged buildings</td>
<td>4.3</td>
</tr>
<tr>
<td>Inventory</td>
<td>5.2</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>3.0</td>
</tr>
<tr>
<td>Fatalities</td>
<td>5.5</td>
</tr>
<tr>
<td>Environment &amp; cultural assets</td>
<td>0.1</td>
</tr>
<tr>
<td>Impact to economy</td>
<td>9.1</td>
</tr>
<tr>
<td>Businesses</td>
<td>7.2</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>0.7</td>
</tr>
<tr>
<td>Rents</td>
<td>1.2</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>35.6</strong></td>
</tr>
</tbody>
</table>

*(in billion USD)*

In Table 1.1 the assessed losses corresponding to the differentiation in Figure 1.1 are summarized. The monetary value assigned to fatalities has been derived using the concept of the Life Quality Index which provides the amount of money which is necessary and affordable for the society to invest into life saving activities (1 million USD per statistical life). Moreover, the compensation of fatalities may also depend on the societal and legal system of the considered country. In case of the failure of the WTC Twin Towers the Victim Compensation Fund compensated the dependents of the fatalities. Based on the compensation scheme an average compensation of 2.0 million USD per fatality can be calculated summarizing to a total cost of compensation 5.5 billion USD.

A major observation from Table 1.1 is that the consequences which would normally be understood as the direct consequences (traditionally forming the basis for optimal design considerations), i.e. the property losses corresponding to the Twin Towers alone plus perhaps rescue and clean-up only amounts to about 12% to 33% of the total loss depending on the assessment of the economic losses.
This observation underlines the significance of the inclusion of possible “follow-up” consequences in
the decision problem of structural design.

### 1.2.2 Decision theoretical framework for optimal design

Based on already existing work in the area of structural optimization, various investigations and
extensions of the theory have been undertaken. In summary optimal design should take basis in a life
cycle benefit optimization. The Bayesian decision theory in conjunction with modern methods of
structural reliability forms a consistent and operational basis for the consideration of the effect of
uncertainties associated with the performance of the structures as well as the consequences
associated with a potential failure or other consequence inducing events. Decisions on optimal design
shall be based on a comparison of expected utility or more specifically expected life cycle benefit.

It has been found that the preferences of individuals or individual groups of stakeholders are not of
relevance as far as decision making on behalf of the society is concerned, which essentially is the
issue when most building structures are considered. What is important though is that all “follow-up”
consequences which might be triggered by an extreme event such as failure of an extraordinary
building structure are included into the formulation of the life cycle benefits. Such “follow-up”
consequences could include negative reactions from the public which again might lead to additional
economical consequences.

A theoretical framework has been developed and implemented within the present project for life
cycle benefit based design of structures. The underlying assumption for the framework is that
structures are systematically rebuilt after potential failures to the same level of safety as before they
failed. Furthermore, it is assumed that the costs associated with future potential failures and
rebuilding activities remain constant over time. The latter is not restrictive in the sense that
predictable changes of such costs may easily be accommodated for by the developed model. Finally,
the developed model also facilitates consideration of the impact of possible structural degradation
and activities for maintenance of the structures over time.

### 1.2.3 Structural reliability and vulnerability assessment

Methods of modern structural reliability analysis, following the Probabilistic Model Code of the Joint
Committee on Structural Safety (JCSS), constitute a methodical cornerstone in the framework of
structural design optimization and in the last decade also a basis for the development of modern
design codes.

Within the present research project the basics of structural reliability are outlined and the link
between structural reliability analysis and the so-called safety factors of design codes is explained.

Some focus is devoted to the special aspects of design of exceptional structures such as high-rise
building structures. It is underlined that any type of structure or load condition falling outside of the
scope of existing design codes should be treated as an exceptional structure. This implies that these
structures are subject to specific reliability assessments, both when they are designed and later on
reassessed.

A framework is developed for the risk based design of structures subjected to ordinary types of loads
as well as extraordinary load events. This framework has the advantage that it directs the focus on the
different possible ways the risk of failure of a structure can be controlled and/or reduced. The
framework is illustrated in Figure 1.2.

Following the framework illustrated in Figure 1.2, three phases are considered. The first phase
concerns the load event itself (exposure of the structure), the second phase the immediate damage
caused by the load event (vulnerability of the structure) and the third phase the ability of the structure
to withstand the immediate damage (robustness of the structure). Following the procedure outlined in
Figure 1.2 it is clear that the risk of failure might be reduced by reducing the exposure, reducing the
vulnerability and increasing the robustness.

The reliability analysis of structural systems which are required to assess the robustness of a structure
can be highly demanding. In general, such analysis must be performed using specialized computer
programs. However, in some cases as described within the present report it is possible to identify
relatively simple systems representing the relevant failure modes of the considered structure. At least for preliminary structural design verification such approaches may be beneficial.

**Figure 1.2** Illustration of suggested framework for the risk assessment of structures.

### 1.2.4 Optimal design of extraordinary building structures

Based on the developed consequence models, the developed framework for optimal design as well as the framework for risk assessment of structures under general load conditions some principal studies concerning optimal design have been performed.

**Figure 1.3**

a) Structural design optimization considering ordinary load situations with and without follow-up consequences and b) extraordinary fire loading. $\varphi$ represents a required percentual increase in design parameters as compared to best present practice.

First, the basic case of optimal design of buildings similar to the WTC towers but considering only normal environmental load effects is investigated. This investigation can thus be understood as an investigation in regard to the required robustness of extraordinary structures for such load cases. The results of the study are summarized in Figure 1.3a. In the figure the total expected life cycle costs are given as a function of an increase of a typical design parameter, such as e.g. the cross-sectional area of a column. The increase is relative to a cross-sectional dimension which would yield a structure with an annual probability of failure equal to $2.4 \times 10^{-5}$, i.e. corresponding to a typical requirement of modern design codes for structures categorized as high consequence structures and high costs of safety improvements, see [86]. The minimum of the illustrated curves indicate the optimal design.
The studies, even though they are only illustrative, clearly show that the effect of follow-up consequences play a significant role and imply that structures of the same type as the WTC Twin Towers should be designed to an annual failure probability one decade lower than structures categorized as high consequence structures in the JCSS Probabilistic Model Code.

For extraordinary building structures of the same type as the WTC Twin Towers the load case of fire is of high relevance. In order to assess the effect of different measures of fire risk reduction a study concerning fire risk reduction addresses the physical behavior of steel structural elements with passive fire protection. A main issue concerns the performance of fire protection of steel structural elements, especially in the case where the fire protection is less than perfect. Such cases are relevant in situations where it can be assumed that the fire protection has been partially damaged, e.g. due to impacts or explosions. The performed investigations have been undertaken using detailed finite element analysis of steel columns and considering different degrees of damage as well as different locations of the damage. The investigations clearly show that whereas the fire protection in an undamaged condition provides an excellent measure of fire risk reduction, very little damage at a critical location will lead to a situation where the effect of the fire protection can be neglected.

In Figure 1.3b the effect of different fire risk reduction measures on the optimal design have been investigated and it is seen that the effect of fire protection on the optimal design can be significant. However, as outlined in the above, the integrity of the passive fire protection is of utmost importance for the risk reduction – implying that a design utilizing fire protection is rather vulnerable in regard to incidents which might lead to fire protection damages.

Finally, with more specific consideration directed to the mode of failure of the WTC Twin Towers an analysis is performed to investigate the possibilities to increase the robustness of severely damaged high-rise building structures to a degree where a progressive collapse can be arrested. Taking basis in the scenario of failure for the WTC Twin Towers two approaches are considered, where the first corresponds to increasing robustness to a degree where no collapse mode will initiate despite a significant damage of the structure at the location of the impact. The second approach is concerned about the situation, where a collapse of one entire floor has taken place and by means of structural measures (e.g. energy dissipation) the energy from the mass of the structure above is taken by the underlying floor and in this way arrests the collapse.

The investigations show that it would be possible to pursue both investigated approaches, even though the first approach is somewhat more cost efficient than the second. However, both approaches seem to be prohibitively expensive. An assessment of the feasibility of the investigated approaches to increase robustness within the framework of structural design optimization has not been undertaken as this would need to address the probability of occurrence of event of the type leading to the failure of the WTC Twin Towers. This issue lies beyond the scope of the present research project.

In summary, the developed framework for optimal design of extraordinary structures has been found operational and forms a consistent framework for the identification of risk reducing design measures. However, it should be underlined that the performed investigations are so far only of an illustrative character, and more detailed and conclusive investigations should be undertaken.
2 Assessment and modeling of consequences

The present chapter focuses on the assessment of consequences which are related to failures of exceptional building structures such as high-rise buildings. In particular it considers the consequences which are associated to the failure of the World Trade Center Twin Towers. To start with, the chapter compares the insured losses of September 11, 2001 with insured losses associated with other major events. Then a scheme is introduced according to which consequence types may be differentiated and which helps to assess consequences before and after the occurrence of an adverse event. Thereafter, a model is formulated for consequence assessment and finally the consequences related to the failure of the World Trade Center Twin Towers are summarized.

2.1 The WTC failure in the context of other insured losses

Early in 2002, the reinsurance company Swiss Re estimated that the insured losses due to the World Trade Center failure [182] were between 30 and 58 billion USD. Therefore, the loss is characterized as one of the highest insured losses ever experienced. Monetarily, this event surpassed such extreme events like the hurricane Andrew, which occurred in 1992, the Northridge Earthquake of 1994 and the European winter storms of 1999. Figure 2.1 shows that the failure of the World Trade Center Twin Towers is the largest economical loss for the insurance industry for more than 30 years [182]. It should be noted that in [182], Swiss Re estimated only the consequences covered by insurances. The actual losses are larger.

Table 2.1 lists the 40 largest insured losses for the period from 1970 to 2001. Here, Swiss Re lists the WTC incident with only its estimated insured property and business interruption losses of 19 billion USD. When the total insured loss of 30 – 58 billion USD is considered, the World Trade Center failure clearly represents the highest insured loss. In [79] it is estimated that the September 11 events will cost the insurance industry about 40 billion USD. Figure 2.3 illustrates the insured losses according to different categories.

The temporal development of the number of fatalities for insured losses is shown in Figure 2.2. Considering this time series, it is seen that the year 2001 was not an exceptional year in terms of fatalities, although 33’000 persons lost their lives in 315 major events. Almost half of the fatalities were caused by 13 earthquakes, whereof 15’000 fatalities are due to the earthquake in Gujarat India. 3’000 fatalities are listed for the failure of the World Trade Center towers. Despite the large number of 3’000 fatalities, the World Trade Center failure is not listed among the 40 worst catastrophes in terms of fatalities, which considers the period from 1970 until 2001, see Table 2.2. However, another event, which occurred in 2001, the earthquake of Gujarat, is the 11th worst event in terms of fatalities. It is seen that the major events listed in this table are dominated by storms, floods and earthquakes, i.e. natural hazards. The largest loss of fatalities in the considered period is due to a cyclone, which occurred in Bangladesh in November 1970 and caused 300’000 fatalities. However, when only terrorist attacks are considered, it is seen that the World Trade Center failure surpasses all previous events in terms of material losses and fatalities, see Table 2.3.
Chapter 2 – Assessment and modeling of consequences

Figure 2.1 Insured losses from 1970 to 2001, from [182].

Table 2.1 The 40 most costly insurance losses from 1970 to 2001, from [182].

<table>
<thead>
<tr>
<th>Insured loss</th>
<th>Victims (^d)</th>
<th>Date</th>
<th>Event</th>
<th>Country</th>
<th>Non-life premium volume (^5)</th>
<th>Loss as % of NL-premium volume (^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29 185</td>
<td>38</td>
<td>23.09.1992</td>
<td>Hurricane Andrew</td>
<td>US, Bahamas</td>
<td>400 238</td>
<td>5.00%</td>
</tr>
<tr>
<td>19 000 (^1)</td>
<td>3 000</td>
<td>11.09.2001</td>
<td>Terrorist attacks on WTC, Pentagon etc.</td>
<td>US</td>
<td>483 481</td>
<td>4.10%</td>
</tr>
<tr>
<td>18 720</td>
<td>60</td>
<td>17.01.1994</td>
<td>Northridge earthquake</td>
<td>US</td>
<td>18 628</td>
<td>4.00%</td>
</tr>
<tr>
<td>15 132</td>
<td>97</td>
<td>27.09.1991</td>
<td>Typhoon Iniki</td>
<td>Japan</td>
<td>113 641</td>
<td>8.90%</td>
</tr>
<tr>
<td>9 221</td>
<td>96</td>
<td>25.01.1990</td>
<td>Winterstorm Daria</td>
<td>France, UK et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>7 164</td>
<td>86</td>
<td>25.12.1990</td>
<td>Winterstorm Lothar over Western Europe</td>
<td>France, UK et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>6 999</td>
<td>61</td>
<td>15.06.1989</td>
<td>Hurricane Hugo</td>
<td>Puerto Rico, US et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>6 774</td>
<td>22</td>
<td>15.10.1987</td>
<td>Storm and floods in Europe</td>
<td>France, UK et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>5 323</td>
<td>13</td>
<td>25.02.1990</td>
<td>Winterstorm Vivian</td>
<td>Western/Central Europe</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>4 201</td>
<td>26</td>
<td>22.05.1999</td>
<td>Typhoon Earl hits south of country</td>
<td>Japan</td>
<td>110 889</td>
<td>3.90%</td>
</tr>
<tr>
<td>3 301</td>
<td>500</td>
<td>20.08.1998</td>
<td>Hurricane Georges</td>
<td>US, Caribbean</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>3 150</td>
<td>33</td>
<td>05.06.2001</td>
<td>Tropical storm Allison; rain, floods</td>
<td>US</td>
<td>483 481</td>
<td>0.70%</td>
</tr>
<tr>
<td>3 094</td>
<td>167</td>
<td>06.01.1988</td>
<td>Explosion on platform Piper Alpha</td>
<td>UK</td>
<td>54 830</td>
<td>5.50%</td>
</tr>
<tr>
<td>2 872</td>
<td>6 425</td>
<td>17.01.1995</td>
<td>Great-Hanshin earthquake in Kobe</td>
<td>Japan</td>
<td>147 881</td>
<td>1.90%</td>
</tr>
<tr>
<td>2 551</td>
<td>46</td>
<td>27.12.1996</td>
<td>Winterstorm Martin over southwest France and Spain</td>
<td>France, E. CT</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2 508</td>
<td>70</td>
<td>10.09.1999</td>
<td>Hurricane Floyd; heavy downpours, flooding</td>
<td>US, Bahamas</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2 440</td>
<td>59</td>
<td>01.10.1990</td>
<td>Hurricane Agnes</td>
<td>US et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2 164</td>
<td>246</td>
<td>03.03.1993</td>
<td>Blizzard, tornadoes</td>
<td>US, Mexico, Canada</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2 019</td>
<td>4</td>
<td>11.09.1992</td>
<td>Hurricane Iniki</td>
<td>US, North Pacific</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 900 (^d)</td>
<td>–</td>
<td>06.04.2001</td>
<td>Hail, floods and tornadoes</td>
<td>US</td>
<td>483 481</td>
<td>0.40%</td>
</tr>
<tr>
<td>1 865</td>
<td>27</td>
<td>23.10.1989</td>
<td>Explosion in petrochemicals plant</td>
<td>US</td>
<td>480 471</td>
<td>0.50%</td>
</tr>
<tr>
<td>1 834</td>
<td>–</td>
<td>12.09.1979</td>
<td>Hurricane Frederic</td>
<td>US</td>
<td>287 185</td>
<td>0.60%</td>
</tr>
<tr>
<td>1 806</td>
<td>39</td>
<td>06.09.1996</td>
<td>Hurricane Fran</td>
<td>US</td>
<td>409 105</td>
<td>0.40%</td>
</tr>
<tr>
<td>1 795</td>
<td>2 000</td>
<td>18.09.1974</td>
<td>Tropical cyclone Fil</td>
<td>Honduras</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 743</td>
<td>138</td>
<td>03.08.1995</td>
<td>Hurricane Luis</td>
<td>Caribbean</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 665</td>
<td>350</td>
<td>10.09.1988</td>
<td>Hurricane Gilbert</td>
<td>Jamaica et al</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 594</td>
<td>29</td>
<td>22.12.1999</td>
<td>Winterstorm Nifel</td>
<td>Western/Northern Europe</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 578</td>
<td>54</td>
<td>03.09.1999</td>
<td>Series of over 70 tornadoes in the Midwest</td>
<td>US</td>
<td>428 291</td>
<td>0.40%</td>
</tr>
<tr>
<td>1 564</td>
<td>500</td>
<td>17.12.1983</td>
<td>Blizzards, cold wave</td>
<td>US, Canada, Mexico</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 550</td>
<td>48</td>
<td>20.10.1991</td>
<td>Forest fires spread to urban areas, drought</td>
<td>US</td>
<td>402 724</td>
<td>0.40%</td>
</tr>
<tr>
<td>1 546</td>
<td>350</td>
<td>02.04.1974</td>
<td>Tornadoes in 14 states</td>
<td>US</td>
<td>222 590</td>
<td>0.70%</td>
</tr>
<tr>
<td>1 475</td>
<td>25.04.1973</td>
<td>Floods in the Mississippi</td>
<td>US</td>
<td>230 345</td>
<td>0.50%</td>
<td></td>
</tr>
<tr>
<td>1 461</td>
<td>15.06.1998</td>
<td>Wind, hail and tornadoes (MN, IA)</td>
<td>US</td>
<td>402 912</td>
<td>0.40%</td>
<td></td>
</tr>
<tr>
<td>1 428</td>
<td>63</td>
<td>17.10.1989</td>
<td>Loma Prieta earthquake</td>
<td>US</td>
<td>480 471</td>
<td>0.40%</td>
</tr>
<tr>
<td>1 319</td>
<td>33</td>
<td>04.08.1970</td>
<td>Hurricane Celia</td>
<td>US, Cuba</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 300</td>
<td>12</td>
<td>19.06.1998</td>
<td>Typhoon Viik</td>
<td>Japan, Philippines</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 257</td>
<td>30</td>
<td>21.09.2001</td>
<td>Explosion in fertiliser factory; 4000 homes destroyed</td>
<td>France</td>
<td>38 338</td>
<td>3.50%</td>
</tr>
<tr>
<td>1 237</td>
<td>46</td>
<td>01.01.1998</td>
<td>Cold spell with ice and snow</td>
<td>Canada, US</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1 219</td>
<td>2</td>
<td>05.06.1995</td>
<td>Wind, hail and flooding (TX, RM)</td>
<td>US</td>
<td>420 819</td>
<td>0.30%</td>
</tr>
<tr>
<td>1 200</td>
<td>2</td>
<td>28.10.1991</td>
<td>Hurricane Grace</td>
<td>US</td>
<td>420 724</td>
<td>0.30%</td>
</tr>
</tbody>
</table>

\(^1\) Excluding liability losses  
\(^2\) Dead and missing  
\(^3\) Premiums in 2000, calculated at 2001 prices  
\(^4\) Figures for natural catastrophes in the US by courtesy of the Property Claims Service (PCS)  
\(^5\) in USD m, at 2001 price levels  
\(^6\) only accounting property and business interruption
The WTC failure in the context of other insured losses

Figure 2.2  Fatalities from 1970 to 2001, from [182].

Table 2.2  The 40 worst events in terms of fatalities in the period from 1970 to 2001, from [182].

<table>
<thead>
<tr>
<th>Victims</th>
<th>Insured loss 1,2,3</th>
<th>Date</th>
<th>Event</th>
<th>Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>350 000</td>
<td>–</td>
<td>14.11.1970</td>
<td>Storm and flood catastrophe</td>
<td>Bangladesh</td>
</tr>
<tr>
<td>250 000</td>
<td>–</td>
<td>28.07.1976</td>
<td>Earthquake in Tangshan (8.2 Richter scale)</td>
<td>China</td>
</tr>
<tr>
<td>138 000</td>
<td>3</td>
<td>29.04.1991</td>
<td>Tropical cyclone Gorky</td>
<td>Bangladesh</td>
</tr>
<tr>
<td>60 000</td>
<td>–</td>
<td>31.05.1970</td>
<td>Earthquake (7.7 Richter scale)</td>
<td>Peru</td>
</tr>
<tr>
<td>50 000</td>
<td>116</td>
<td>21.06.1990</td>
<td>Earthquake in Gilan</td>
<td>Iran</td>
</tr>
<tr>
<td>25 000</td>
<td>–</td>
<td>17.12.1988</td>
<td>Earthquake in Armenia</td>
<td>Armenia, ex-USSR</td>
</tr>
<tr>
<td>20 000</td>
<td>–</td>
<td>16.09.1978</td>
<td>Earthquake in Tabas</td>
<td>Iran</td>
</tr>
<tr>
<td>20 000</td>
<td>–</td>
<td>13.11.1985</td>
<td>Volcanic eruption on Nevado del Ruiz</td>
<td>Colombia</td>
</tr>
<tr>
<td>22 000</td>
<td>233</td>
<td>04.02.1976</td>
<td>Earthquake (7.4 Richter scale)</td>
<td>Guatemala</td>
</tr>
<tr>
<td>19 118</td>
<td>1063</td>
<td>17.08.1999</td>
<td>Earthquake in Izmit</td>
<td>Turkey</td>
</tr>
<tr>
<td>15 000</td>
<td>100</td>
<td>26.01.2001</td>
<td>Earthquake (moment magnitude 7.7) in Gujarat</td>
<td>India, Pakistan</td>
</tr>
<tr>
<td>15 000</td>
<td>106</td>
<td>29.10.1999</td>
<td>Cyclone 05B devastates Orissa state</td>
<td>India, Bangladesh</td>
</tr>
<tr>
<td>15 000</td>
<td>–</td>
<td>01.09.1978</td>
<td>Flooding following monsoon rains in northern parts</td>
<td>India</td>
</tr>
<tr>
<td>15 000</td>
<td>530</td>
<td>19.09.1985</td>
<td>Earthquake (6.1 Richter scale)</td>
<td>Mexico</td>
</tr>
<tr>
<td>15 000</td>
<td>–</td>
<td>11.08.1979</td>
<td>Dyke burst in Morvi</td>
<td>India</td>
</tr>
<tr>
<td>10 800</td>
<td>–</td>
<td>31.10.1971</td>
<td>Flooding in Bay of Bengal and Orissa state</td>
<td>India</td>
</tr>
<tr>
<td>10 000</td>
<td>234</td>
<td>15.12.1999</td>
<td>Flooding, mudslides, landslides</td>
<td>Venezuela, Colombia</td>
</tr>
<tr>
<td>10 000</td>
<td>–</td>
<td>25.05.1985</td>
<td>Tropical cyclone in Bay of Bengal</td>
<td>Bangladesh</td>
</tr>
<tr>
<td>10 000</td>
<td>–</td>
<td>20.11.1977</td>
<td>Tropical cyclone in Andrah Pradesh and Bay of Bengal</td>
<td>India</td>
</tr>
<tr>
<td>9 500</td>
<td>–</td>
<td>30.05.1990</td>
<td>Earthquake (6.4 Richter scale)</td>
<td>Maharashtra, India</td>
</tr>
<tr>
<td>9 000</td>
<td>543</td>
<td>22.10.1998</td>
<td>Hurricane Mitch in Central America</td>
<td>Honduras, Nicaragua, et al,</td>
</tr>
<tr>
<td>8 000</td>
<td>–</td>
<td>16.08.1976</td>
<td>Earthquake on Mindanao</td>
<td>Philippines</td>
</tr>
<tr>
<td>6 425</td>
<td>2 872</td>
<td>17.01.1995</td>
<td>Great Hanshin earthquake in Kobe</td>
<td>Japan</td>
</tr>
<tr>
<td>6 304</td>
<td>–</td>
<td>05.11.1991</td>
<td>Typhoons Thelma and Uring</td>
<td>Philippines</td>
</tr>
<tr>
<td>5 000</td>
<td>1 044</td>
<td>05.03.1987</td>
<td>Earthquake</td>
<td>Ecuador</td>
</tr>
<tr>
<td>5 000</td>
<td>426</td>
<td>23.12.1972</td>
<td>Earthquake in Managua</td>
<td>Nicaragua</td>
</tr>
<tr>
<td>5 000</td>
<td>–</td>
<td>30.06.1976</td>
<td>Earthquake in West-Iran</td>
<td>Indonesia</td>
</tr>
<tr>
<td>5 000</td>
<td>–</td>
<td>10.04.1972</td>
<td>Earthquake in Fars</td>
<td>Iran</td>
</tr>
<tr>
<td>4 500</td>
<td>–</td>
<td>10.10.1980</td>
<td>Earthquake in El Asnam</td>
<td>Algeria</td>
</tr>
<tr>
<td>4 375</td>
<td>–</td>
<td>21.12.1987</td>
<td>Ferry Dona Paz collides with oil tanker Victor</td>
<td>Philippines</td>
</tr>
<tr>
<td>4 000</td>
<td>–</td>
<td>30.05.1998</td>
<td>Earthquake in Takhar</td>
<td>Afghanistan</td>
</tr>
<tr>
<td>4 000</td>
<td>–</td>
<td>15.02.1972</td>
<td>Storms and snow in Ardekan</td>
<td>Iran</td>
</tr>
<tr>
<td>4 000</td>
<td>–</td>
<td>24.11.1976</td>
<td>Earthquake in Van</td>
<td>Turkey</td>
</tr>
<tr>
<td>4 000</td>
<td>–</td>
<td>02.12.1984</td>
<td>Accident in chemical plant in Bhopal</td>
<td>India</td>
</tr>
<tr>
<td>3 840</td>
<td>6</td>
<td>01.11.1997</td>
<td>Typhoon Linda</td>
<td>Vietnam et al.</td>
</tr>
<tr>
<td>3 800</td>
<td>–</td>
<td>08.09.1992</td>
<td>Flooding in Punjab</td>
<td>India, Pakistan</td>
</tr>
<tr>
<td>3 656</td>
<td>327</td>
<td>01.07.1998</td>
<td>Flooding along Yangze River</td>
<td>China</td>
</tr>
<tr>
<td>3 400</td>
<td>1063</td>
<td>21.05.1999</td>
<td>Earthquake in Nantou</td>
<td>Taiwan</td>
</tr>
<tr>
<td>3 200</td>
<td>–</td>
<td>16.04.1978</td>
<td>Tropical cyclone</td>
<td>Réunion</td>
</tr>
</tbody>
</table>

1 Dead or missing
2 Excluding liability losses
3 In USD m, at 2001 price levels
Table 2.3  The 10 most severe terrorist attacks in the period from 1970 to 2001, from [182].

<table>
<thead>
<tr>
<th>Victims</th>
<th>Insured Loss</th>
<th>Date</th>
<th>Event</th>
<th>Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>at least 3000</td>
<td>19 000</td>
<td>11.09.2001</td>
<td>Terror attack against WTC, Pentagon and other buildings</td>
<td>USA</td>
</tr>
<tr>
<td>300</td>
<td>23.10.1993</td>
<td>Bombing of US Marine barracks and French paratrooper base in Beirut</td>
<td>Lebanon</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>6 12.03.1993</td>
<td>Series of 13 bomb attacks in Mombasa</td>
<td>Germany</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>21.12.1993</td>
<td>Bomb attack on government building in Oklahoma City</td>
<td>USA</td>
<td></td>
</tr>
<tr>
<td>166</td>
<td>23.11.1996</td>
<td>Bomb attack on government building in Oklahoma City</td>
<td>USA</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>13.09.1999</td>
<td>Bomb explosion destroys apartment block in Moscow</td>
<td>Russia</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>04.06.1991</td>
<td>Arson in arms warehouse in Addis Ababa</td>
<td>Ethiopia</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>31.01.1999</td>
<td>Bomb attack on Geylinco House in Colombia</td>
<td>Sri Lanka</td>
<td></td>
</tr>
</tbody>
</table>

1 Dead or missing
2 Excluding liability losses

2.2 Categorization of consequences

Structural failures of exceptional infrastructural facilities such as high-rise buildings imply losses and consequences of various types and extends. In order to assess consequences systematically, it is meaningful to categorize consequences into different types. First of all, consequences may be differentiated into consequences to humans, environment and not least economic consequences. In addition, also cultural assets may be damaged or destroyed.

Furthermore, consequences are differentiated into direct and indirect consequences. Direct consequences are directly related to the considered facility and event; indirect consequences occur as a result of direct consequences. It is clear that this differentiation underlies a subjective consideration of what can be seen as a direct or indirect consequence.

To ease the process of consequence assessment, consequences are categorized into different types such that the consequences are mutually exclusive. This allows for obtaining the total consequences by adding up the different consequence types.

Figure 2.4 shows the considered types of consequences, which are considered in the following chapters. It is seen that the consequences may be differentiated into material and economic losses, cultural asset losses and consequences to persons and environment. The material and economic consequences comprise rescue and clean-up costs, the loss and damage to the WTC towers itself and to the surrounding buildings and facilities. Furthermore, they include the damaged or destroyed inventory which was contained in the buildings and facilities. In addition, the impact on the economy is accounted for in terms of business interruption and lost rents. Furthermore, lost cultural assets form a material loss, which can be evaluated, but they also constitute a cultural loss, which currently cannot be quantified. Moreover, environmental consequences may have effects on plants, animals and not least persons. Finally, also consequences due to fatalities have to be considered.

The considered categories of consequences aim to address all consequences relevant for failures of high-rise buildings. Although extensive, they should not be seen as exhaustive. Hazard pointers may be used in order to identify additional consequences types, which may be relevant, see [52].
2.3 Modeling of consequences

At first, consequences occur as an amount of lost, damaged or destroyed assets such as goods, services and lives. Examples hereof are the amount of destroyed or damaged office space, roads, water supply systems, sewage systems, etc. The amount of lost, damaged or destroyed assets defines also the intensity of the consequence. When assessing risks related to the failure of a building, the amount of lost, damaged or destroyed assets can be known precisely or with a negligible variability, e.g. lost building’s gross floor area, when the structure fails completely; however, they may also be associated with uncertainty, e.g. when the structure fails only partially. The amount of lost, damaged or destroyed assets may be summarized in the random vector

\[ \mathbf{a} = (a_1, a_2, \ldots, a_n)^\top, \]

where \( n \) is the number of considered consequences. For some variables \( a_i \) it may be sufficiently accurate to use expected values rather than describing the individual variables by their distribution type and the corresponding parameters. \( a_i \) may also be described by a damage factor \( d_i \). This approach is especially useful, when an inventory list is already at hand. Then a damage factor \( d_i \) may be associated to each element \( \varepsilon_i \) of the inventory list, which is represented by the vector \( \varepsilon \). The damage factor is larger or equal to zero and smaller or equal to one. \( a_i \) may be obtained by \( a_i = d_i \varepsilon_i \) or when \( \mathbf{D} \) is a diagonal matrix with diagonal elements \( d_i \), by \( \mathbf{a} = \mathbf{D}\varepsilon \). For specific hazards it is possible to interrelate the damage factor with hazard specific measures, see e.g. Voortman [200] and HAZUS [115].

All consequences should be expressed in monetary units. This is easy to achieve for all material losses. When fatalities are considered the Life Quality Index and the Societal Life Saving Costs may be used as a basis for assessing the corresponding monetary loss to society, see B.4.4.

Consider a consequence type \( i \) such as clean-up costs, property losses etc. and let \( c_{i,j} \) be the unit costs associated with \( a_j \), then \( \mathbf{c}^\top \mathbf{a} \) expresses the corresponding monetary value of the consequence. Uncertainty may be related to the unit costs, as well. Then, \( \mathbf{c}_i = (c_{i,1}, c_{i,2}, \ldots, c_{i,n}) \) is a random vector. However, it is usually sufficiently accurate to use expected values.
In order to account for the variation of costs with respect to time, the function $\varphi_t(\cdot)$ is introduced. It considers variation of costs with time, such as price rise due to inflation. In order to measure the real price rise instead of nominal price rise, this function has to be adjusted for inflation. $\varphi_t(\cdot)$ is subject to uncertainty as well and may be described by a random process, if appropriate. Finally, $\Phi_{r,i}(t)$ the monetary consequences of type $i$ of an incident occurring at time $t$, e.g. structural failure, can be expressed as:

$$C_{\Phi_{r,i}}(t) = \varphi_t(t) \cdot e^T a \cdot \varphi_t(t) \cdot e^T \Delta e.$$  

(2.1)

The total consequences due to an incident at time $t$ are obtained by summing up all mutually exclusive types of consequences.

$$C_F(t) = m \sum_{i} C_{\Phi_{r,i}}(t)$$

(2.2)

$C_F(t)$ is a function of random variables and therefore random itself. The factor $m$ accounts for the multiplier effect, which models a reduced consumption of the affected persons, businesses and organizations of the society. An introduction to the multiplier effect is given in Annex C.

In a risk analysis all consequences which are relevant and meaningful for the underlying decision making have to be identified. If a priori the relevance of a consequence type can not be estimated, it should be considered in a first approach. A sensitivity study will then reveal its relevance by showing its influence on the optimum decision.

Equation 2.1 and 2.2 provide a basis to assess any type of consequences. However, in the following subsection, the assessment of business interruption is considered in more detail.

### 2.3.1 Business interruption

Losses due to business interruption are consequences which may result from an adverse event. Businesses may cease their activity in order to provide access to rescue teams or they may close due to safety reasons. When a structure fails, businesses may not only lose office space, they even may lose the basis for doing business. Generally, the economic loss due to business interruption is assessed by the lost gross domestic product (GDP) or respectively the lost value added, which would have been produced during the interruption. An introduction on the GDP and the value added is given in Annex C.

The value added accounts for the benefit generated by the enterprises, the income of employees, taxes, rents and capital costs. It is rather easy to assess, if data and information are available. A great advantage of modeling consequences by means of the GDP and the value added is that such models are independent of the tax, social and juridical system of the considered country. This approach permits the assessment of losses, without addressing who and to what extent will bear it, by:

$$C_{F,q}(t) = \int_{\Omega} \int_{\omega} \varphi_{q}(\tau) \cdot d_{q}(\mathbf{x}, \tau) \cdot \Delta g_{q}(\mathbf{x}) \cdot n_{q}(\mathbf{x}) \cdot d\mathbf{x} \cdot d\tau.$$  

(2.3)

In Equation 2.3, $C_{F,q}(t)$ is the economical loss due to business interruption associated with an event occurring at time $t$. This is obtained by integration over a considered geographical area $\Omega$ and time. $\varphi_{q}(\tau)$ considers the time variation of the GDP. $d_{q}(\mathbf{x}, \tau)$ is the damage factor, which characterizes the damage according to the spatial and temporal distribution. $d_{q}(\mathbf{x}, \tau)$ is smaller than or equal to one and is larger or equal to zero; when $\tau$ goes to infinity, $d_{q}(\mathbf{x}, \tau)$ approaches zero. $\Delta g_{q}(\mathbf{x})$ is the value added produced per time unit at the location $\mathbf{x}$ and $n_{q}(\mathbf{x})$ considers the interrelation of the business at $\mathbf{x}$ with the remaining economy outside of the considered area $\Omega$. For instance, [122] indicates that every job in the financial sector supports two other jobs. $n_{q}(\mathbf{x})$ is a factor accounting for the value added by these interrelated jobs. These jobs must not be located inside of $\Omega$ to avoid double counting. $n_{q}(\mathbf{x})$ is larger or equal to one.

In Equation 2.3, $\Delta g_{q}(\mathbf{x})$ can be interpreted as the productivity, which is the value added per area. In general, the productivity can be calculated for each production factor, namely land, labor and capital. Thereby for instance, the value added per employee or facility is obtained.
2.4 Consequences due to the failure of the World Trade Center Twin Towers

The present section gives a short summary of the consequences resulting from the failure of the World Trade Center Twin Towers. A more comprehensive overview with reference to the data sources is provided in Annex B.

The World Trade Center towers WTC1 and WTC2 were built in the late 60’s and early 70’s. At that time the towers cost 900 million USD. On September 11, 2001 both towers collapsed after a terrorist attack. The succeeding rescue efforts and clean-up were finished earlier than expected and were with 1.7 billion USD cheaper than expected. A reconstruction of the towers would presumably cost 4.7 billion USD. The replacement costs of the other destroyed buildings are estimated to 2.0 billion USD and the repair costs of the damaged buildings are expected to costs 4.3 billion. The inventory stored within these buildings, is estimated to 5.2 billion USD. The damage of the surrounding infrastructure is estimated to 3.0 billion USD. This includes the damage to the PATH station and the MTA subway as well as the damage to power, gas, steam and telecommunication facilities.

Consequences due to fatalities are often seen as the most difficult to assess. The monetary value assigned to fatalities has been derived using the concept of the Life Quality Index which provides the amount of money which is necessary and affordable for the society to invest into life saving activities (1·million USD per statistical life). Moreover, the compensation of fatalities may also depend on the societal and legal system of the considered country. In case of the failure of the WTC Twin Towers the Victim Compensation Fund compensated the dependents of the fatalities. Based on the compensation scheme an average compensation of 2.0 million USD per fatality can be calculated summarizing to a total cost of compensation of 5.5 billion USD.

Due to the considerable reconstruction period, the failure of the World Trade Center will imply an estimated loss in rents of 1.2 billion. Formally, business interruption can be assessed simply; however, this consequence type inheres much uncertainty. The losses are estimated as low as 7.2 billion USD and range up to 64.3 billion USD. The difference arises from uncertainties, which are inherent to the modeling of the economic development. Influential is also the expert judgment on how much of the economic loss may be allocated to the events of September 11 and how much to the economic recession. Lastly, the analyzed reports, which assess the impact on the economy, vary in the period for which they estimate the economic impact. Economic losses associated with the infrastructural facilities are estimated to 0.7 billion USD.

Table 2.4 Summary of consequences.

<table>
<thead>
<tr>
<th>Consequence type</th>
<th>Scenario Low</th>
<th>Scenario High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rescue &amp; clean-up</td>
<td>1.7</td>
<td>1.7</td>
</tr>
<tr>
<td>Property</td>
<td>19.2</td>
<td>19.2</td>
</tr>
<tr>
<td>WTC Twin Towers</td>
<td>4.7</td>
<td></td>
</tr>
<tr>
<td>Other destroyed buildings</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>Damaged buildings</td>
<td>4.3</td>
<td></td>
</tr>
<tr>
<td>Inventory</td>
<td>5.2</td>
<td></td>
</tr>
<tr>
<td>Infrastructure</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>Fatalities</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Environment &amp; cultural assets</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Impact to economy</td>
<td>9.1</td>
<td>66.2</td>
</tr>
<tr>
<td>Businesses</td>
<td>7.2</td>
<td>64.3</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>Rents</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>35.6</strong></td>
<td><strong>92.7</strong></td>
</tr>
</tbody>
</table>

20 million USD are committed to establish a health registry, which will follow up the health condition of up to 200'000 people due to environmental impacts. However, the 20 million USD will not cover future compensation payments to these people. For instance, more than one thousand
firemen filed lawsuits against the City of New York with a total claim corresponding to 12 billion USD. Moreover, artworks and cultural assets were lost. The art works situated within the WTC were insured to 100 million USD. However, this does not account for the non-reproducible loss of masterpieces e.g. of Miró, Rodin, Picasso, as well as the cultural assets, which were stored in the Greek orthodox St. Nicholas church.

Depending on the economical impact assessment the total consequences associated with the failure of the World Trade Center ranges between 35.6 to 92.7 billion USD, which corresponds to 7.6 to 19.7 times the reconstruction costs of the Twin Towers. Rescue and clean-up costs constitute 36% of the towers reconstruction costs, whereas the total loss of property including the towers costs 4.09 times the reconstruction costs. Consequences due to fatalities amount to 1.17 times the reconstruction costs and the environmental consequences and destroyed cultural assets only correspond to 3% of the towers’ reconstruction costs because neither possible future compensation of rescue workers nor the non-reproducible loss of cultural assets is considered. Finally, the economic consequences are estimated to be in the interval between 1.94 to 14.1 times the reconstruction costs, see also Table 2.4.
3 Decision theoretical framework

Civil engineering structures, such as high-rise buildings and other infrastructural facilities, such as roads, tunnels, bridges, sewage water systems, airports etc. constitute the backbone of modern societies. Civil engineering structures permit, enable or facilitate ordinary or extraordinary activities of individuals, enterprises, administrations and other organizations of the society. For instance, an infrastructural network permits to transport persons and goods by individual and public transport using streets, tunnels, bridges, railways, airports, harbors etc. It also provides space for business activities such as industrial production, retail or offices for consulting, finance, insurance, real estate, education, etc. Other important sectors, which depend on civil engineering facilities, are energy supply and telecommunication. Hence, civil engineering structures are part of a well developed infrastructural network and their elements may be interrelated and dependent on each other.

![Risk Based Structural Design Diagram](image)

**Figure 3.1 Framework for risk based structural design.**

During the last decades it has become increasingly recognized that the design and maintenance of structures must be seen within a life cycle costs and benefit framework. Such an approach is provided if the concept of risk is introduced in the process of structural design. Using a risk based approach to structural design, different design alternatives may be evaluated considering the whole...
life cycle of the structure. Finally, the optimal design which maximizes the life cycle benefit may be identified and chosen.

Figure 3.1 illustrates this approach. In the top of the figure, there is the structural design of a structure. It influences the reliability of the structure to withstand loads, e.g. from permanent loads, live loads, fire, wind, snow earthquake. On the other hand, the design may influence the consequences associated to adverse events. Here, consequences are distinguished into consequences to humans, damage to the qualities of the environment and economic losses and consequences to cultural assets. A risk based structural design may evaluate the utility of different structural designs and identify the optimum.

The present chapter outlines the decision theoretical framework for risk based design. To start with, the objective of structural design is identified, and approaches to optimum design and their interrelations are discussed. Furthermore, the life cycle benefit is introduced and a consistent framework for the consideration of follow-up consequences is presented. Finally, the chapter closes with a discussion on interest rates and sustainability.

3.1 Objective of structural design

In the following it is assumed that the owner, operator or otherwise responsible of a structure aims to achieve the maximum expected benefit during the anticipated service life of the structure. This implies that due consideration must be given to all benefits and potential costs during the service life of the structure. Risk assessments taking basis in the Bayesian decision theory provide a consistent framework for life cycle optimal design of structures. In Figure 3.1, the framework is illustrated showing that structural reliability assessments as well as consequence assessments form the main ingredients in optimal design considerations. Using this framework, it is possible both to identify whether a design is feasible and whether a design is optimal.

\[
LCB = C_{\text{var}} + C_{\text{fix}} + C_O + C_I + C_M + C_{\text{Rep}} + C_F + C_D + C_{\text{Rev}} = LCC + C_{\text{Rev}}
\]  

(3.1)

\[
LCC = C_{\text{var}} + C_{\text{fix}} + C_O + C_I + C_M + C_{\text{Rep}} + C_F + C_D
\]  

(3.2)

The mathematical formulation for life cycle benefit is given in Equation 3.1. Here, \( LCB \) is the life cycle benefit. It comprises the construction costs, which are dependent \( C_{\text{var}} \) and independent \( C_{\text{fix}} \) of the structural design. In addition, it consists of the costs for the operation \( C_O \), maintenance \( C_M \), inspection \( C_I \), repair \( C_{\text{Rep}} \) and decommissioning \( C_D \). Furthermore, the consequences due to structural failures \( C_F \) and the revenue \( C_{\text{Rev}} \), i.e. the income, have to be considered. The revenue is not considered in Equation 3.2, which is the formulation of the life cycle costs \( LCC \). In Equation 3.1, both the operation costs and revenue are considered; however, sometimes these two components are considered together by the net revenue.

It should be noted that if Equation 3.1 is applied, then the revenue will be inserted with positive values and costs with negative values. When the life cycle costs are calculated, then in practice, the costs are generally inserted in Equation 3.2 as positive values. In this case, the \( LCC \) have to be minimized. The extremal points of \( LCB \) and \( LCC \) are obtained by differentiation with respect to the design variable(s). From Equation 3.1 and 3.2 it is seen that their derivatives are equal, if the revenue \( C_{\text{Rev}} \) is not a function of the design variable(s). In this case, both approaches yield the same optimum design and a minimization of the life cycle costs also maximizes the life cycle benefit.

However, it should be noted that for some types of structures constructed for a special purpose and with a short duration such as structures built for the exploitation of resources it is important to include the revenue in the optimization problem. In this case the revenue can be highly influenced through the design parameters and influence the optimal decision. Optimal design of such structures is studied in [92].

From Equation 3.1 and 3.2 it is seen that the concept of life cycle benefit is the most general approach to express a structure’s utility in monetary terms. By means of the Life Quality Index (LQI) and the Societal Life Saving Costs (SLSC), consequences due to fatalities may also be included in the framework of optimal design of structures.
Much uncertainty is associated with the costs, consequences, revenues and the reliability of the structure; therefore, the life cycle benefit is uncertain. According to decision theory, decision should be made according to expected values. Hence, the objective of structural design should be the maximization of the expected life cycle benefit \( B \).

\[
B = E[LCB] = E[C_{C,\text{var}}] + E[C_{C,\text{fix}}] + E[C_{O}] + E[C_{M}] + E[C_{\text{Rep}}] + E[C_{F}] + E[C_{D}] + E[C_{\text{Rev}}]
\] (3.3)

### 3.2 Expected life cycle benefit

The expected life cycle benefit for structural design was principally introduced by Rosenblueth and Mendoza \[168\]. Almost 30 years later the introduced idea was taken up by Rackwitz \[161\], who proposed it as the basis of code making. In Rosenblueth and Mendoza \[168\] it is assumed that failures occur as realizations of a Poisson process. Furthermore, the authors consider the events and decisions after the failure of a structure. The question arises whether a failed structure will be reconstructed or not. Two reconstruction strategies are considered. The first reconstruction strategy assumes that a failed structure will not be reconstructed and that the activity associated to the structure will be stopped. According to the second reconstruction strategy, a failed structure will always be reconstructed. When the latter reconstruction strategy is used, Rosenblueth and Mendoza \[168\] consider the revenues and consequences associated to the succeeding structures within the anticipated duration of the activity supported by the structures. In Rackwitz \[161\], the formulation of life cycle benefit is extended using basic concepts of the renewal theory. This allows for the consideration of non-stationary failure processes, which result from non-stationary loads or non-stationary resistances, e.g. due to deterioration such as fatigue and corrosion.

In \[51\] the optimization problem is formulated as a pre-posterior problem in accordance with the Bayesian decision theory. By using event/decision trees, this permits to account for the effects of maintenance, inspections and repairs. Furthermore, by use of Bayes theorem, probabilities may be updated, when additional information is available, see also \[165\].

In Kübler and Faber \[93\] these two developments were integrated and the expected life cycle benefit \( B(z,i,d) \) was formulated in a way that the optimal design together with the optimal inspection and maintenance plan can be assessed. Following this, the expected life cycle benefit can be written as:

\[
B(z,i,d) = E_{X,\Theta,\Psi,i,d}[LCB(z,i,d(\Psi,F(i)),\Theta,\Psi)]
\] (3.4)

In Equation 3.4, \( B(z,i,d) \) is the expected life cycle benefit and is obtained when the expectation operation \( E[\] \) is performed. The expected life cycle benefit is a function of the variables \( z \), \( i \) and \( d \). \( z \) is a vector containing design variables such as section modulus, \( i = (t_1, l_1, q_1)^T \) is the vector containing the parameters of the inspection plan, namely \( t_1 \) inspection times, \( l_1 \) the vector of inspection locations and \( q_1 \) the vector containing the different possible inspection qualities. The uncertain inspection results are represented by the random vector \( \Psi \). \( d(\Psi,F(i)) \) is a decision rule defining the rehabilitation strategy following an inspection result \( \Psi = \Psi \) or a failure of the structure at time \( t \). \( \Theta \) characterizes the uncertain performance of the structure, and finally, \( X \) is a vector containing the basic random variables required for the probabilistic modeling of loads, material characteristics and costs. The optimal structural design together with the best inspection and maintenance plan \( z^*,i^*,d^* \) are obtained as the values maximizing the expected life cycle benefit \( B \).

\[
B^* = B(z^*,i^*,d^*) = \max_{z,i,d} B(z,i,d)
\] (3.5)

### 3.2.1 Events, decision trees and probabilities

For the purpose of assessing the life cycle benefit or costs, decision/event trees may be utilized, see e.g. \[52\]. Within these logical trees, all events, which may occur during the design lifetime, should be represented in a logical order. If \( E_i \) is the \( i \)th component of the tree with \( j_i \) possible states, then \( E_{i,k_i} \) is the event that the \( j_i \)th component is in state \( k_i \). If \( n_E \) is the number of components, then the total number of paths is given by
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\[ n_U = \prod_{j=1}^{x_j} j_i. \]  \hfill (3.6)

When the design of structures is considered, the occurrence in time of these events and their logical order depend on the structural design \( z \), the inspection and maintenance plan \( i \) and the reconstruction strategy \( d \). A single path \( U_k \) is defined as the intersection of events.

\[ U_k = \left\{ E_{1,k_1} \cap \ldots \cap E_{n_k,k_{n_k}} \right\} \]  \hfill (3.7)

The probability associated to this event is

\[ U_k = P\left( E_{1,k_1} \cap \ldots \cap E_{n_k,k_{n_k}} \right) \]  \hfill (3.8)

Failure and reconstruction of a structure are usually rather costly events. Figure 3.2a illustrates all possible events for a given time interval, e.g. time between inspections. It is illustrated that the initial structure might survive this time period, indicated by the event \( F_1 \), which is equal to the event \( S \) indicating a survival. However, the structure may fail, as well. This case is indicated as \( F_2 \). In case the structure fails, it might be reconstructed or not, represented by the events \( \text{Rec} \) and \( \bar{\text{Rec}} \), respectively. Generally, this decision will be made according to its economical feasibility. If the structure is not reconstructed, the activity supported by the structure is ceased. If the structure is reconstructed, the activity is continued. However, the new structure might fail as well, denoted by the event \( F_3 \). Then a reconstruction decision has to be made, again. Following this scheme, an infinite number of failures is possible. For the case, when it can be assumed that a failed structure will always be reconstructed, the event tree can be simplified, see Figure 3.2b.

![Figure 3.2](image)

Figure 3.2 Possible failures and reconstructions within a time interval.

On the right side of Figure 3.2a and 3.2b it is illustrated that the infinite number of events may be summarized by three events, namely \( \{ S \} \) the survival of the initial structure, \( \{ S \cap \text{Rec} \} \) the event where a reconstructed structure survives and finally \( \{ \bar{S} \cap \text{Rec} \} \) refers to the case where a failed structure is not reconstructed.

Figure 3.3a shows an event tree representing all events which might occur between two inspections. The three events \( \{ S \} \), \( \{ S \cap \text{Rec} \} \) and \( \{ \bar{S} \cap \text{Rec} \} \) are illustrated; it is seen that when a structure is not reconstructed, then the activity, which is associated to the structure, will be stopped. When an inspection is performed at \( t_{i,j} \), the event \( I \) represents the event that an indication of deterioration is found. In addition, \( \text{Rep} \) denotes that a repair action is implemented. The complementary events are \( I \) and \( \bar{\text{Rep}} \), respectively.
### 3.2.1.1 Reconstruction and repair strategy

A decision rule $d(\Psi, F(t))$ is introduced to define the decisions made with regard to repair or reconstruction depending on the specific outcome of the performance of the structure (i.e. failure at time $t$) or the result of the inspections $\Psi = \psi$.

As already mentioned, for the case of a structural failure two decision alternatives exist. The first alternative is that a failed structure will be removed and the activity it supported will be ceased, whereas the second alternative is the reconstruction of the failed structure. This decision must be performed according to its economical feasibility. However, civil engineering structures, such as high-rise buildings and other infrastructural facilities are important for the infrastructural network, which is provided to the society. Therefore, it may be assumed that a reconstruction is always the optimal decision and that failed structures will be systematically reconstructed. Furthermore, it is assumed that in case of a reconstruction, the reconstructed structure will have the same reliability as the preceding one. This reconstruction strategy corresponds to the event tree given in Figure 3.2b.

The assumed reconstruction strategy is also supported by the experiences from the World Trade Center. For instance, after September 11, 2001, the PATH station and tunnel was reconstructed and repaired, the collapsed subway tunnel was reconstructed, and the completely failed WTC7 is currently under reconstruction.

Furthermore, decision rules for repair actions can be defined. This simplifies the event tree shown in Figure 3.3b even more and keeps it manageable and numerically tractable. For further derivations the following assumptions are made:

1) When a structure fails between inspections, it will be reconstructed. A consecutive inspection will not give an indication of deterioration, and no repair will be made.

2) When a structure survives the time between inspections, an inspection might give an indication of deterioration, which automatically will lead to a repair of the structure. No repair will be performed if the inspection result does not indicate deterioration.

Implementing these assumptions, Figure 3.3c is obtained. Finally, Figure 3.3d illustrates a full event tree considering three inspections. The top event is the construction with design $z$.

### 3.2.2 Evaluation of probabilities

In probabilistic terms, a path in the event tree represents an intersection of events. This intersection can be expressed as a sequence of conditional events in time, where the structural reliability and the probability of indication of deterioration are conditioned on the events, which occurred before. For instance, if a repair or a reconstruction is performed at time $t_0$, the initial reliability will be reestablished, and the events after this point in time will be influenced by this event; however, they may be assumed to be independent of the events, which occurred before $t_0$. Generally, information
about performed inspections has to be used to express the conditional probability of getting a specific inspection result at a later point in time. Furthermore, it can be used to update the structural reliability; in the following however, this possibility is not pursued.

It is assumed that failures are independent events and follow a non–homogenous Poisson process. The reliability function for a specific path in the event tree is given by Equation 3.9 and the probability that the structure does not survive the time interval \( \Delta t_j = [t_{i,j-1}, t_{i,j}] \) by Equation 3.10.

\[
R(t \mid U_k) = \exp \left[- \int_0^{t-t_0} v^*(r) \, dr \right]
\]

(3.9)

\[
P(\bar{S}(t_{i,j}) \mid U_k) = R(t_{i,j-1} \mid U_k) - R(t_{i,j} \mid U_k)
\]

(3.10)

In these equations, \( v^*(t) \) is the mean out-crossing rate, and \( t_0 \) is the last point in time when the initial reliability was reestablished (either by repair or reconstruction). \( t_0 \) depends on the path \( U_k \). By means of a possible realization, Figure 3.4 illustrates the influence of a repair and reconstruction action at \( t_{i,j-1} \), \( t_f \) and \( t_{i,j} \) on the structural resistance \( b(t) \) and the mean out-crossing rate \( v^*(t) \). At \( t_{i,j-1} \), an inspection is carried out at which deterioration is indicated and a subsequent repair rehabilitates the resistance. At \( t_f \), the structure fails, e.g. due to an extreme loading larger than the resistance \( b(t) \). The structure is reconstructed at \( t_f \) and inspected at \( t_{i,j} \). The inspection shows no deterioration and therefore, no repair will be done.

![Figure 3.4](image-url)

The probability of an indication of deterioration at time \( t_{i,j} \), \( P(I(t_{i,j})) \) has to be considered for three cases. The first case considers that the structure does not survive the time interval \( \Delta t_j = [t_{i,j-1}, t_{i,j}] \), then \( t_0 \) is smaller than \( t_{i,j-1} \) but larger than \( t_{i,j} \). According to assumption 1) an inspection after a reconstruction will not indicate deterioration.

\[
P(I(t_{i,j}) \mid Rec(t_{i,j}) \cap \bar{S}(t_{i,j})) = 0
\]

(3.11)

The second case considers a repair made at \( t_{i,j-1} \), then \( t_0 = t_{i,j-1} \) and the conditional probability of indication for deterioration is

\[
P(I(t_{i,j}) \mid Rep(t_{i,j-1}) \cap I(t_{i,j-1})) = P(I(t_{i,j-1} \mid I(t_{i,j-1}))
\]

(3.12)

Finally, the case is considered, where the structure is neither repaired nor reconstructed. In this case \( t_0 \) is less than \( t_{i,j-1} \). The probability to get an indication of deterioration is conditioned on the results of the most resent inspection results. If \( \bar{I}(t_{i,k}) \) is the first inspection result not leading to a repair, and if no repair or reconstruction is performed between \( t_{i,k} \) and \( t_{i,j} \), then the conditional probability of obtaining an indication of deterioration at time \( t_{i,j} \) is

\[
P(I(t_{i,j} \mid t_{i,k}) \cap I(t_{i,j} \mid t_{i,k}) \cap \ldots \cap I(t_{i,j} \mid t_{i,k})) = \frac{P(I(t_{i,j} \mid t_{i,k}) \cap \bar{I}(t_{i,j-1} \mid t_{i,k}) \cap \ldots \cap \bar{I}(t_{i,j} \mid t_{i,k}))}{P(I(t_{i,j-1} \mid t_{i,k}) \cap \ldots \cap \bar{I}(t_{i,j} \mid t_{i,k}))}.
\]

(3.13)

The event of indication of deterioration \( I(t) \) at a certain point in time \( t \) can be expressed in terms of a limit state function \( g_I(x, t) \). By means of structural reliability methods, such as FORM/SORM or Monte Carlo simulations, this function can be evaluated probabilistically. The probability of indication of deterioration is formulated as \( P(I(t)) = P(g_I(X, t) \leq 0) \) and \( P(I(t)) = P(g_I(X, t) > 0) \) is the probability of no indication. [93] shows that if the same inspection method is used, the limit state
functions at different points in time are highly correlated and if the limit state function $g_l(x,t)$ is a nonincreasing function for constant $x$ and for increasing $t$, Equation 3.13 may be simplified to:

$$P\left(\bigcap_{i=1}^{m} D_i \right) = \frac{P\left(\bigcap_{i=1}^{m} D_i \right) - P\left(\bigcap_{i=1}^{m} D_i \right)}{1 - P\left(\bigcap_{i=1}^{m} D_i \right)}.$$  

**3.2.3 Assessment of expected costs and revenues**

**3.2.3.1 Discounting**

After having assessed the probabilities, the expected values may be evaluated. For the purpose of comparison of different costs and revenues, which occur at different points in time, a continuous discounting function $r(t)$ is introduced. The discount factor $\gamma$ is related to the yearly discount rate $\gamma'$ by $\gamma = \ln(1 + \gamma')$.

$$r(t) = \exp(-\gamma t)$$  

In Section 3.4 the issue of discounting and sustainability is further discussed.

**3.2.3.2 Expected construction costs**

The cost of construction is a function of $p$ design variables $z_j$ stored in the vector $z$. Often, the costs of construction is modeled as a linear function of the design variables, where $C_{C_{fix}}$ are the fixed construction costs, which are not influenced by the design variables. Recalling that $X$ is a vector containing the basic random variables required for the probabilistic modeling of loads, material characteristics and the costs, the uncertainties related to the costs may also be accounted for when the uncertain variables $C_{C_{fix}}$ and $C_{C_{j}}$ are included in the vector $X$.

$$E[C] = E_X \left[ C_{C_{fix}} + \sum_{j=1}^{p} C_{C_{j}} z_j \right]$$  

**3.2.3.3 Expected revenues**

For the assessment of the expected revenue, an integration over the design lifetime has to be performed. The integrand is the product of the discounted revenue and the probability that the revenue is obtained. For the considered reconstruction strategy, the probability that a revenue is obtained – this is the probability that the activity is not ceased – is unity for any point in time. Then the expected value of the revenue is given by Equation 3.17. $C_{Rev}(\tau)$ is the time variant revenue weighted by the discounting function $r(\tau)$. Again, uncertainties related to the cost models $C_{Rev}(\tau)$ and $r(\tau)$ may be accounted for. It is seen that in this case, the expected revenue is independent of the structural design variable and the inspection and maintenance plan. Hence, with the considered reconstruction strategy, the minimization of the life cycle benefit maximizes also the life cycle benefit. A situation, where this is not the case can be found in [92].

$$E[C_{Rev}] = E_X \left[ \int_{0}^{T} C_{Rev}(\tau) \ r(\tau) \ d\tau \right]$$  

**3.2.3.4 Expected inspection costs**

The expected inspection costs are the sum of the single inspection costs $C_I(\tau)$ weighted by the discounting function $r(\tau)$. The inspection times are collected in the vector $t_i$. $t_{i,j}$ is the $j$th inspection time of the inspection plan $i$ and $n_i$ is the number of inspections. $\delta(t)$ is Dirac’s delta function.

$$E[C_I] = E_X \left[ \sum_{j=1}^{n_i} \int_{0}^{T} \delta(t - t_{i,j}) \ C_I(\tau) \ r(\tau) \ d\tau \right]$$  

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3.2.3.5 Expected repair costs

The expected value of the repair costs can be expressed as follows.

\[
E[C_{\text{rep}}] = E_X \left[ \sum_{k=1}^{n_U} \sum_{j=1}^{n_{\text{rep},k}} \int_0^\tau b(\tau - t_{\text{rep},j}) \ C_{\text{rep}}(\tau) \ r(\tau) \ P(U_k) \ d\tau \right] \tag{3.19}
\]

This equation sums over all possible paths \( U_k \) in the event tree, where \( n_U \) is the number of paths. The second sum considers the repair times \( t_{\text{rep}} \) for a given path \( U_k \) and the associated probability \( P(U_k) \). Here, \( t_{\text{rep},j} \) is the time of the \( j^{th} \) of \( n_{\text{rep}} \) repairs associated with path \( U_k \). \( C_r(\tau) \) are the repair costs, which may vary with respect to time.

3.2.3.6 Expected decommissioning costs

For the underlying reconstruction strategy, where failed structures will be reconstructed successively, decommissioning will be made at the end of the design lifetime. Then the expected value is given as follows.

\[
E[C_{\text{D}}] = E_X \left[ C_{\text{D}}(T) \ r(T) \right] \tag{3.20}
\]

\( C_{\text{D}}(t) \) are the time varying decommissioning costs and \( T \) is the design lifetime of the activity. Again, uncertainties related to the cost may be accounted for in the expectation operation.

3.2.3.7 Expected failure costs

The expected value of the failure costs is given by the following equation.

\[
E[C_F] = E_X \theta \left[ \sum_{k=1}^{n_U} \sum_{j=1}^{n_{\text{rep},k}} \int_\tau^\infty C_F(\tau) \ r(\tau) \ P(U_k) \ h(\tau|U_k) \ d\tau \right] \tag{3.21}
\]

Again, this equation sums over all paths \( U_k \) and integrates the failure costs \( C_F(\tau) \) piecewise over each time interval between inspections over the whole design lifetime. The failure costs are weighted by the discounting function \( r(\tau) \) and \( P(U_k) \) the probability of the path event. \( h(\tau|U_k) \) is the renewal density conditioned on the events of the path. Equation 3.22 shows that if no failure occurs within \( \Delta t_{i,j} = [t_{i,j}, t_{i,j+1}] \), the conditional renewal density is zero, otherwise it is the renewal density divided by the probability that the structure does not survive \( \Delta t_{i,j} \). \( t_{i,t} \) is again the most recent point in time, at which the initial reliability was reestablished.

\[
h(t|U_k) = \begin{cases} \frac{h(t-t_{i,t})}{P(S(t_{i,j+1}|U_k))}, & \text{if } S \in \Delta t_{i,j} \\ 0, & \text{if } S \in \Delta t_{i,j} \end{cases}, \quad \text{where } \Delta t_{i,j} = [t_{i,j}, t_{i,j+1}] \quad \text{and } t_{i,t} = \text{most recent point in time,} \tag{3.22}\]

3.2.4 Renewal density and backward recurrence-time

According to [33], the renewal density \( h(t) \) is the probability of one or more renewals within an infinitesimal time interval. In structural reliability theory, a structural failure can be interpreted as a renewal.

\[
h(t) = \lim_{\Delta t \to 0} \frac{P(\text{one or more renewals in } [t, t+\Delta t])}{\Delta t} = \sum_{i=1}^{\infty} f_i(t) \tag{3.23}\]

When, \( f_i(t) \) is the probability density function of the \( i^{th} \) renewal (failure), then the renewal density can be expressed by means of an infinite sum over the probability density functions of the \( i^{th} \) renewal. The probability density function of the \( i^{th} \) renewal can be calculated by the following convolution integral.

\[
f_i(t) = \int_0^t f_{i-1}(t-\tau) f_i(\tau) d\tau, \quad i = 2, 3, \ldots \tag{3.24}\]
For all practical purposes, the failure process of civil engineering structures can be assumed to follow a non-homogenous Poisson process. In this case the probability density function for the first passage time is given by

$$f_1(t) = v^*(t) \exp \left[ - \int_0^t v^*(t') dt' \right].$$

(3.25)

In this equation, $v^*(t)$ is the mean out-crossing rate, which can be evaluated for time variant loads and resistances, see e.g. [14], [86] and [52].

$$v^*(t) = \lim_{\Delta t \to 0} \frac{E\left[ N_{i,t+i,\Delta} \right]}{\Delta t}$$

(3.26)

Generally, the mean out-crossing rate is conditional on the vector of non ergodic variables $\mathbf{R}$ and the vector of ergodic sequences $\mathbf{Q}$. The expectation operation can be performed together with the computation of $v^*(t)$ as described by Schall et al. [172]. By means of the out-crossing rate, the probability density function for the first passage time, the probability density function of the time to a $i$th renewal/failure and the renewal density can be evaluated numerically. Sufficient accuracy of the renewal density is reached after a few terms of $f_1(t)$. Convergence is reached more rapidly for large coefficients of variation (CoV’s) of the first passage time as also indicated in [161]. For very large CoV’s, the renewal density might be approximated with sufficient accuracy by the out-crossing rate and the asymptotic renewal density $1/E[T]$.

If a repair or a reconstruction is made at $t_{li}$ the initial reliability is reestablished. The renewal density function considering this is equal to the initial renewal function shifted backwards in time by $t_{li}$.

$$h(t|\text{Rec or Rep at } t_{li}) = h(t-t_{li})$$

(3.27)

Only $t_{li}$ is of interest because repairs or reconstructions performed before do not influence the reliability after $t_{li}$. For a certain point in time $t$ and for a given path $U_k$, $t_{li}$ can be determined as follows.

$$t_{li} = \max\left\{ t_{rep}^i, t_{rec}^i, 0 \right\} \quad \text{and} \quad t_{li} \leq t$$

(3.28)

In this expression $t_{rep}$ is the vector of repair times and $t_{rec}$ is the vector of times of reconstruction. Both are uniquely determined by the path $U_k$.

### 3.2.4.1 Backwards recurrence-time

In the foregoing it is mentioned that the repair times $t_{rep}$ and the reconstruction times $t_{rec}$ are required in order to facilitate updating of the renewal density and the probability of obtaining an indication of deterioration. The repair times are known, if the time difference between inspection, indication of deterioration and the successive repair is neglected. However, the times at which failure and reconstructions occur are uncertain. For a given path however, the intervals in which a structure does not survive are known. By means of the backwards recurrence-time $W$, see [33], the life time distribution of the current structure can be assessed. The backwards recurrence-time $W$ is the time measured backwards from $t$ to the most recent failure event at or before $t$, see Figure 3.5.

![Figure 3.5 Realization of the backwards recurrence-time $W$.](image)

The probability density function of the recurrence time is given by the probability $h(t-w)$ that within an infinitesimal time interval at $t-w$ a renewal (failure with a consecutive reconstruction) occurs multiplied with the probability $R(w)$ that no renewal occurs (the new structure survives) until $t$. 

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\[ f_w(w) = h(t - w)R(w) \]  

Equation (3.29)

For non-homogenous Poisson processes the probability density function given in Equation 3.29 may be evaluated numerically. In [93] an approximation for practical applications may be found.

3.3 Follow-up consequences and risk aversion

The Bayesian decision theory as described, e.g. by Raiffa and Schlaifer [165] together with the axioms proposed by von Neumann and Morgenstern [111] form a well accepted rational and operational framework for decision making in general and for the specific case of engineering decision making. It is also adapted as the basis for the Probabilistic Model Code [86] of the Joint Committee on Structural Safety (JCSS).

In the practical application of decision theory it is often argued that risk perception needs to be taken into account in the formulation of utility function. The reason for this is that the utility function must be able to represent the “real” behavior of the decision maker in regard to his or her preferences for a given situation. The amount of research invested into the experimental investigation and the mathematical modeling of the behavior of human decision makers is vast, see e.g. Pratt [159], Arrow [6] and Kahneman and Tversky [88]. Whereas this research clearly points to the basic characteristics of the perception of risks under different conditions, it also points to a whole set of problems related to the consistent formulation of utility functions as summarized in Camerer and Weber [15] and also discussed in Maes and Faber [96]. For this reason the axioms of utility theory proposed by von Neumann and Morgenstern [111] have been heavily disputed during the last decades and various competing formulations suggested, see e.g. [88].

For the modeling of the effect of risk perception, one can differentiate between two different situations, namely 1) the situation where the purpose is to predict and represent the behavior and the attitudes of decision makers and 2) the situation where the purpose is to provide support for rational decision making, e.g. for society.

Faber and Maes [55] proposed a framework for the appropriate representation of risk perception in risk based decision making in engineering problems, i.e. where the purpose is to provide support in the decision making for owners of engineering activities, owners of structures and authorities. The basic premise of [55] is that the risk aversion intrinsic to non-linear utility functions can almost always be explained by the non-inclusion of certain “follow-up” consequences. “Follow-up” consequences are, generally speaking, triggered by extreme losses, such as excessive business losses, loss of reputation or other indirect or so-called intangible losses. The non-inclusion of such losses occurs either voluntarily or involuntarily. In principle, the use of an appropriate non-linear utility functions and the inclusion of “follow-up” consequences may be mathematically equivalent and therefore may lead to identical decisions; however, only the latter approach leads to risk-consistent rational decision making.

As outlined in Maes and Faber [55], it can be stated that most decision makers and risk engineers would agree to the basic principle of ranking alternatives \( A \) based on their expected utility \( E[U(A)] \) following von Neumann and Morgenstern [111].

\[ E[U(A)] = \sum_{i=1}^{n_o} u(A,O_i)P(O_i | A) \]  

Equation (3.30)

In Equation 3.30, \( n_o \) is the number of possible discrete outcomes associated to alternative \( A \), \( P(O_i | A) \) is the probability that the outcome \( O_i \) will take place conditioned on the alternative \( A \), and \( u(A,O_i) \) is the utility associated with \( O_i \) and \( A \). Whereas Equation 3.30 is based on a discrete set of outcomes, the extension to continuous outcomes is straightforward.

Depending on the situation at hand, decision makers reportedly [88] feel uncomfortable with the direct application of Equation 3.30, due to principally two reasons: either the decision maker is uncertain about the assessment of the utility \( u(A,O_i) \) or, he/she is uncertain about the assessment of the probabilities \( P(O_i | A) \). When decision making in civil engineering is seen as being equivalent to participate in a game with nature as the main opponent, see also Ditlevsen and Madsen [37], then the uncertainty about the assessment of the probabilities and the utilities corresponds to not really knowing the rules of the game. In principle the effect of misjudging the utility associated with a
particular outcome is comparable to misjudging the probability of the outcomes’ occurrence. This in turn leads to both over- and underestimation of the expected utility. However, given that all relevant outcomes and all uncertainties – aleatoric, as well as epistemic uncertainties, see [54] – have been included into the formulation of the expected utility, risk adverse or risk prone behavior seems irrational and also inappropriate. Despite this fact, in practical risk based decision making only the direct consequences are usually taken into account. The experience from the WTC failure clearly shows that follow-up consequences may be a multiple of direct consequences. It is clear that a simplified utility, which does not consider any follow-up consequences gives rise to a risk perception requiring an artificial, nonlinear and risk averse utility function. The effect of inappropriately modeled uncertainties onto the risk perception follows analogously.

In order to consider aleatoric, as well as epistemic uncertainties together with the consequences, which also account for follow-up consequences, Faber and Maes introduced following expression, see [55].

\[
E[U(A)] = E_e \left[ \sum_{i=1}^{n_O} u(A,O_i) P(O_i|A,e) + \sum_{j=1}^{m} u_{Fu}(A,O_j) P(O_j|A,e) \right]
\]  

(3.31)

In Equation 3.31, \( u(A,O) \) are the utilities associated with \( O \) and \( A \) and \( P(O_i|A,e) \) are aleatoric probabilities conditional on the alternative \( A \) and the outcome of the epistemic uncertainties \( e \). An additional term takes into account “follow-up” consequences. In this term \( m \) is the number of different combinations \( O_j \) of one or more of the \( n_O \) outcomes associated with the alternative \( A \). \( P(O_j|A,e) \) is the probability that this combination occurs and \( u_{Fu}(A,O_j) \) is the corresponding marginal utility. When the epistemic uncertainty may be integrated out, then Faber and Maes [55] obtain the following expected utility.

\[
E[U(A)] = \sum_{i=1}^{n_O} u(A,O_i) P(O_i|A) + E_e \left[ \sum_{j=1}^{m} u_{Fu}(A,O_j) P(O_j|A,e) \right]
\]  

(3.32)

Compared to Equation 3.30, this equation is reformulated to explicitly account for follow-up consequences. For the specific case when failure consequences are assessed, the expected utility \( E[U(A)] \) may be replaced with the expected failure costs \( E[C_F(A)] \) and the utility \( u(A,O) \) can be substituted by the consequences \( C(A,O) \), which depend on the alternative \( A \) and the outcome \( O \). Furthermore, the follow-up consequences \( u_{fu}(A,O_j) \) may be written as \( C_{fu}(A,O_j) \).

### 3.4 Interest rates and sustainability

It is generally appreciated that risk based decision making is highly influenced by the interest rate applied for discounting of benefits and costs occurring in the future. Net present values are established as the difference between the economical growth rate, corrected for variations in the population, and the rate of inflation. In many cases it is seen that owners and responsible of structures choose the interest rates to be applied in decision making based on political considerations, i.e. based on preferences interrelated to the considered organization and boundary conditions in regard to financing possibilities. When investments on behalf of society are considered it is clear that a firm and rational basis for the assessment of net present values is needed. Furthermore, this basis should reflect the preferences of society rather than reflecting organizational policies.

New considerations and concepts on sustainable decision making see Faber and Nishijima [56] and Rackwitz et al. [164], give directions on how to establish interest rates which will lead to sustainable decisions, if applied in decision analysis as it is presently being carried out. In the following the basic principles underlying sustainable decision making are shortly outlined following Faber and Nishijima [56].

#### 3.4.1 Theoretical framework for sustainable decision making

Decision making in the field of civil engineering often takes basis in optimization problems of the following form
Chapter 3 – Decision theoretical framework

\[
\max_{D(0)} B(D(0)), \quad (3.33)
\]

where \( B \) is the total expected life cycle benefit and \( D(0) \) is a vector of decision alternatives where the parameter 0 indicates that the decision alternatives, which indeed might involve activities in the future, are decided upon at time \( t = 0 \), i.e. the time of the decision by the present decision maker (present generation). In this formulation of the decision problem, utility is implicitly set equal to monetary benefits. In accordance with existing formulations for life cycle costing, the total expected life cycle benefits for the reference period \( T \) are assessed as:

\[
B(D(0)) = \int_0^T b(t, D(0)) \gamma(t) dt, \quad (3.34)
\]

where \( b(t, D(0)) \) is the expected benefit per time unit and \( \gamma(t) \) is a function capitalizing the benefits possibly gained in the future into net present value. The decision problem as stated in Equation 3.33 and Equation 3.34 might be solved within the framework of the pre-posterior decision analysis as outlined e.g. in Raiffa and Schlaifer [165].

If the principle of equity of decision makers over time, see Faber and Nishijima [56] is invoked, this implies that the benefit function given in Equation 3.34 must be extended with the preferences, i.e. the benefits of the future decision makers. The principle is illustrated in Figure 3.6. In this figure it is indicated that exploitation of resources and the benefits achieved by this can be transferred between decision makers at different times. In principle if a generation decides to exploit a resource, which is recyclable only to a certain degree, a part of the benefit achieved by this generation must be transferred to the next generation. In monetary terms this part must correspond to the recycling costs plus compensate for the loss of the non-recyclable resource. The latter compensation could e.g. be in terms of invested research aiming to substitute the resource with fully recyclable resources. Also costs, e.g. associated with the maintenance of structures, may be transferred between decision makers at different times. In Figure 3.6 the joint decision maker is assumed to make decisions for the best of all (also future decision makers) with equal weighing of the preferences of the present and all future decision makers. Following this principle we have to sum the benefits of present and future decision makers as it is seen from their perspective (e.g. in accordance with the state of the world at their point in time and capitalized to their point in time). The interest rate \( \gamma(t) \) to be considered for the individual future decision makers should represent both the preference to spend money \( \rho \) early rather than late (preference discounting) and the growth of the wealth of society (per capita) \( \delta \) e.g. \( \gamma = \rho + \delta = 0.03 + 0.02 = 0.05 \% \) per annum as discussed in Bayer and Cansier [9] and Rackwitz et al. [164]. The societal growth of wealth can and should, however, also be taken into account to compensate for the improved economical capabilities of future decision makers. The benefits of future decision makers must thus be weighed (reduced) in the overall decision problem with the discounting factor \( \delta(t) \) which is given as:

\[
\delta(t) = e^{-\delta \tau}, \text{ where } \delta' = \ln(1+\delta). \quad (3.35)
\]

The benefit function for the joint decision maker (see Figure 3.6) can then be written as

\[
B(D(T)) = \sum_{i=1}^{n} \delta(t_1) \int_{t_i}^{t_{i+1}} b_\xi(\tau, D(t_i), t_i) \gamma(\tau - t_i) d\tau, \quad (3.36)
\]

where \( b_\xi(\tau, D(t_i), t_i) \) is the benefit function and \( D(T) = \{ D(t_i); t_i \in \{ t_1, t_2, \ldots, t_n \} \} \) are the possible decision alternatives for the decision maker at time \( t_i \). \( n \) is the number of considered decision makers.

Based on Equation 3.36, optimization of decisions may now be undertaken considering to the best of knowledge the preferences of future decision makers as well as the way resources and economical means might be transferred over time. It should be noted that the way decision analysis is usually being applied at present, e.g. for the purpose of optimization of design and inspection and maintenance planning is in contradiction of the formulation given in Equation 3.36. This is because the real mechanisms of the transfer of e.g. monetary benefits and costs are not taken properly into account in the decision analysis.
3.4.2 Sustainable discounting

Consider the design optimization problem based on minimum life cycle costs. Following Equation 3.36 and e.g. Rackwitz et al. [164] the total expected life cycle costs $E[LCC(z)]$ (omitting the benefit and assuming systematic rebuilding at failures) are given as:

$$E[LCC(z)] = C_C(z) + (C_C(z) + C_F) \frac{1 - e^{-\gamma \tau}}{1 - e^{-\delta \tau}}. \quad (3.37)$$

in the case that the occurrence of failures follows a Poisson process, under the assumption that $\tau = t_{i+1} - t_i$ is the duration of each generation for all generations and $n \to \infty$. $C_C(z)$ is the design cost where $z$ represents a design parameter, $C_F$ is the cost of failure. If $\gamma = \delta$, implying that time preference discounting is neglected, Equation 3.38 is reduced to the classical result where no special considerations are given to the preferences of future decision makers.

The effective discounting factor is introduced as the discounting factor which, if applied to a decision problem not accounting explicitly for future generations (as in Equation 3.34), yields the same total expected utility as when accounting explicitly for the preferences of the future decision makers (as in Equation 3.36). It can be shown that the effective discounting factor $\gamma^*$ satisfies the following relationship.

$$\gamma^* = \frac{1 - \exp(-\delta \tau)}{1 - \exp(-\gamma \tau)} \gamma. \quad (3.38)$$

It should be noted that $\gamma^* = \gamma$ for $\tau \to \infty$ and $\gamma^* = \delta$ for $\tau \to 0$.
assumed that $\delta = 0.02$ and the length of a generation is about 25 years the effective interest rate to be applied (if decisions are based on Equation 3.34) is around 0.022. This is significantly less than $\gamma = 0.05$ usually adapted in such decision analysis and leads to decisions where a much larger proportion of costs are taken initially at the point of the decision as compared to the costs which are postponed into the future.
4 Structural reliability and vulnerability assessment

Reliability of structures has been a research area devoted significant interest over especially the last 3-4 decades. In the present chapter an overview of the most important aspects of structural reliability is provided and a framework for risk assessment of structures subject to extraordinary load events is suggested. As an introduction a short overview on the safety philosophy utilized in codified design of civil engineering structures is given followed by a categorization of exceptional structures. Thereafter, an introduction to structural reliability analysis is given, where the issues of uncertainty modelling, the analysis of failure probabilities and probability updating are introduced. After that, it is shown how modern structural design codes implement the concepts of structural reliability theory in order to assure a target level of reliability for civil engineering structures. Furthermore, a framework for the risk based design and assessment of structures is provided which is especially suited for the considerations of rare or extraordinary load events. Finally two examples are given considering the vulnerability of structures to fire loads and the robustness of high-rise buildings due to progressive collapse.

4.1 Codified design of structures

During the past century considerable effort has been devoted to the development of a rational basis for the design of structures, which resulted in a number of modern design codes [17], [24], [31], [151], [4] and not least the Eurocodes [21]. The modern codes aim to ensure the economical design, construction and operation of structures in compliance with assumed operational conditions and given requirements for the safety of personnel and the environment. The development of the modern design codes has been based on the principles of economical decision analysis and modern reliability methods, see e.g. [86] and [85]. For the verification of the structural reliability in regard to the relevant failure modes, the modern design codes provide a set of so-called design equations relating the design resistance of the structure for the individual failure modes and the corresponding design load-effects. Due to the fact that loads and resistances are subject to uncertainties, design values for resistances and load effects are introduced to ensure an adequate level of reliability. Design values for resistances are introduced as a characteristic value of the resistance divided by a partial safety factor, which generally is larger than one. Design values for load effects are introduced as characteristic values multiplied by a partial safety factor, which is larger than one. In order to take into account the effect of simultaneously occurring loads, so-called load combination factors are multiplied on one or more of the variable loads. Generally, load combination factors are smaller than one. Design codes utilizing this approach for achieving an adequate level of safety are often referred to as Load and Resistance Factor Design (LRFD) codes.

For the purpose of ensuring a practically applicable design basis, the design codes, which comprise of design equations, characteristic values, partial safety factors and load combination factors have been calibrated for “normal” structures. Here normal structures are structures of usual dimensions and design and which are built with well-known materials and which are constructed and are maintained using established procedures in order to achieve an adequate and homogeneous level of reliability. However, this implies that structures which are not “normal” in the above-mentioned sense fall beyond the application area of the design codes. These types of structures may be seen as being exceptional structures. For such structures the design verifications and for that matter any “fit for purpose” assessment must take basis in reliability assessments for the specific structure.
In the following, first a categorization of different types of exceptional structures is introduced and discussed. Thereafter, the basic principles for the reliability verification of such structures are outlined.

### 4.2 Categories of exceptional structures

Traditionally exceptional structures are usually associated with structures which fulfill new purposes; they are of extreme dimensions or have innovative designs, see e.g. Figures 4.1 and 4.2.

![Figure 4.1](image1.png) ![Figure 4.2](image2.png)

**Figure 4.1** Examples of structures of extreme dimensions. On the left the Great Belt Link under construction and the Sears Tower on the right.

However, in accordance with the definition outlined in the foregoing, exceptional structures include all structures falling beyond the application area of the design and assessment codes. When categorizing such structures, it is useful to differentiate between new structures, i.e. structures to be designed and existing structures, i.e. structures, which for some reason are subject to a reliability assessment.

For new structures, exceptional structures include structures, which:

1. fulfill new purposes or of exceptional dimensions and innovative designs.
2. are built by using new materials or innovative combinations of materials.
3. are constructed and maintained according to new methods and strategies.
4. are subjected to unusual loads and load combinations.
5. are subjected to unusual environmental exposures.
6. are associated with extreme consequences in case of failure.
7. are especially difficult to decommission.

For existing structures, exceptional structures include structures, which:

1. have been designed according to out dated standards.
2. exhibit unforeseen degrees of deterioration.
3. have been subjected to accidental damages.
4. have been subject to extreme loads or environmental exposures.
5. are subject to changed operational conditions.
6. are unexpectedly decommissioned.

In principle, structure specific reliability assessments must be made for all the above-mentioned structures. The engineering profession has recognized this fact to some extent but only within the last decade has the problem been approached in a more systematic and consistent way by using the principles of decision analysis and structural reliability theory. Whereas the principles of decision analysis are outlined in Chapter 3, the basic principles for reliability assessments will be outlined in the following according to [53].
4.3 Principles of structural reliability analysis

The overall aim of structural reliability analysis is to quantify the reliability of structures under consideration of the uncertainties associated with the resistances and loads. The structural performance is assessed by means of models based on physical understanding and empirical data. Due to idealizations, inherent physical uncertainties and inadequate or insufficient data the models themselves are subject to uncertainty, as well as the parameters which are used to describe the model. Parameters may be material characteristics such as the yield strength or load characteristics, e.g. live load. Structural reliability theory takes basis in the probabilistic modeling of these uncertainties and provides methods for the quantification of the probability that the structures do not fulfill the performance criteria.

4.3.1 Uncertainty modeling

The uncertainties which must be considered are the physical uncertainty, the statistical uncertainty and the model uncertainty. The physical uncertainties are typically uncertainties associated with the loading environment, the structure’s geometry and the material properties. The statistical uncertainties arise due to incomplete statistical information, e.g. due to a small number of materials tests. Finally, the model uncertainties must be considered to take into account the uncertainty associated with the idealized mathematical descriptions used to approximate the structure’s actual physical behavior. The probabilistic modeling of uncertainties highly rests on a Bayesian statistical interpretation of uncertainties. This implies that the uncertainty modeling utilizes and facilitates both the incorporation of statistical evidence about uncertain parameters and uncertainties assessed by expert judgment. Modern methods of structural reliability analysis and risk analysis allow for a very general representation of these uncertainties ranging from non-stationary stochastic processes and fields to time invariant random variables, see e.g. [101] or [52]. In most cases it is sufficient to model the uncertain quantities by random variables with given distribution functions and distribution parameters estimated on basis of statistical and/or subjective information. In the JCSS Probabilistic Model Code [86] an almost complete set of probabilistic models is given, which covers most situations encountered in practical engineering problems.
4.3.2 Probabilities of failure

The performance of a structure can normally be expressed in terms of limit state equations \( g(x) \), which are related to so-called failure events \( F \) by following equation.

\[
F = \{ g(x) \leq 0 \} 
\]  

(4.1)

Here, the components of the vector \( x \) are realizations of \( X \), the so-called basic random variables. The later vector represents all relevant uncertainties influencing the probability of failure, whereby the basic random variables must be able to represent all types of uncertainties that are included in the analysis.

Having established probabilistic models for the uncertain variables, the problem remains to evaluate the probability of failure corresponding to a specified reference period. However, also other non-failure states of the considered component or system may be of interest, such as excessive damage, unavailability, etc. In general, any state which may be associated with consequences in terms of costs, loss of lives and impact to the environment is of interest. In the following, however, for simplicity these states are not differentiated.

When the failure event is defined by means of a limit state function, then the probability of failure may be determined by the following equation:

\[
P_F = P(g(X) \leq 0) = \int_{g(x) \leq 0} f_X(x) \, dx. 
\]  

(4.2)

\( P_F \) is the probability of failure, which is identical to the probability that the limit state function \( g(x) \) shows values smaller or equal to zero. Mathematically, this is evaluated by means of the integral on the right hand side. The integration is performed over the domain, for which \( g(x) \) is smaller or equal to zero. In that equation \( f_X(x) \) is the joint probability density function of the random variables \( X \). This integral is illustrated in Figure 4.3 as a volume integral of a two dimensional joint probability density function. Here, the two uncertain variables are the load and the resistance, and failure is defined as the event when the load exceeds the resistance. Despite its simple appearance, it is generally nontrivial to solve the integral and numerical approximations are expedient. Various methods for the solution of Equation 4.2 have been proposed including numerical integration techniques, Monte Carlo simulation and First and Second Order Reliability Methods (FORM/SORM). Numerical integration techniques very rapidly become inefficient for increasing dimension of the vector \( X \). Therefore, they are in general irrelevant for practical applications. The first developments of First and Second Order Reliability Methods (FORM/SORM) took place almost 30 years ago with pioneering work performed by Basler [8], Cornel [32] and Hasofer and Lind [77]. Since then, these methods together with advanced Monte Carlo simulation techniques have been refined and extended significantly. By now they form the most important methods for reliability evaluations in structural reliability theory. For the most common practical purposes the problem of estimating probabilities may be considered as solved. Several commercial computer codes have been developed for FORM/SORM and simulation analysis; the methods are widely used in practical engineering problems and not least for code calibration purposes, see e.g. STRUREL [166] and Proban [38].

![Figure 4.3](image-url) The failure probability integration problem for two dimensions.
4.3.3 Reliability updating

When existing structures are assessed, a significant difference can and should be considered. Different to the design of new structures, the assessment of existing structures may utilize available information and update probabilities. The probabilistic assessment of existing structures is treated in detail in [87]. Examples of information related to existing structures, which may be available or might be made available at a given cost, are:

1) The survival of the structure
2) Material characteristics from different tests
3) Measurements of the actual geometry
4) Damages and deterioration
5) Capacity by proof loading
6) Static and dynamic response to controlled loading

In the assessment of existing structures such new information can be taken into account and combined with the prior probabilistic models. Prior probabilistic models are models which have been formulated before the new information was available. Using updating techniques, so-called posterior probabilistic models may be obtained. These models can then be used as an enhanced basis for the reassessment decision analysis.

Given an inspection result of a quantity \( I \), which is an outcome of a functional relationship between several basic variables, probabilities may be updated. Therefore, the definition of conditional probabilities may be utilized and finally the relevant failure probabilities may be updated, which is indicated by \( P(F|I) \).

\[
P(F|I) = \frac{P(F \cap I)}{P(I)} \tag{4.3}
\]

Inspection or test results relating directly to realizations of random variables may be used for updating. This is done by assuming the distribution parameters of the distributions, which are used in the probabilistic modeling, to be uncertain themselves. New samples or observations of realizations of the random variables are then used to update the probability distribution functions of these distribution parameters.

Assume that a random variable \( X \) has the probability distribution function \( F_X(x;q) \) and density function \( f_X(x;q) \), where \( q \) are the distribution parameters. Furthermore, assume that one or more of the distribution parameters, e.g. the mean value and standard deviation of \( X \) are uncertain themselves and are modeled by the random variables \( Q \) with probability density function \( f_Q(q) \). Then the probability distribution function for \( Q \) may be updated on the basis of observations of \( X \), which are represented by the vector \( \hat{x} \). The general scheme for the updating is

\[
f_Q(q|\hat{x}) = \frac{f_Q(q)L(q|\hat{x})}{\int_{-\infty}^{\infty} f_Q(q)L(q|\hat{x}) dq}, \tag{4.4}
\]

where \( f_Q(q) \) is the probability density function for the uncertain parameters \( Q \) and \( L(q|\hat{x}) \) is the likelihood of the observations or the test results \( \hat{x} \). Here, ‘’ denotes posterior and ‘’ indicates the prior probability density functions of \( Q \). The likelihood function \( L(q|\hat{x}) \) may be readily determined by taking the density function of \( X \) in \( \hat{x} \) with the parameters \( q \). For discrete distributions the integral is replaced by the summation symbol.

The observations \( \hat{x} \) may not only be used to update the distribution of the uncertain parameters \( Q \) but also to update the probability distribution of \( X \). The updated probability distribution function for \( X \) is often called the predictive distribution or the Bayes distribution. The predictive distribution may be assessed by following integration:
In Raiffa and Schlaifer [165] and Aitchison and Dunsmore [3] a number of closed form solutions to the posterior and the predictive distributions can be found for special types of probability distribution functions known as the natural conjugate distributions.

### 4.4 Reliability and partial safety factors

In code based design formats such as the Eurocodes, design equations are prescribed for the verification of the capacity of different types of structural components in regard to different modes of failure. The typical format for the verification of a structural component is given as design equations with partial safety factors $\gamma$ and characteristic values $x$, such as:

$$ g_d(\gamma, x_k) = z R_k / \gamma_M - \left( \gamma_G G_k + \gamma_Q Q_k \right) > 0. $$

(4.6)

Here, $R$ is the resistance, $G$ indicates the permanent load and $Q$ is the variable load. The index $k$ indicates characteristic values of the associated variable, whereas the $\gamma$’s represent partial safety factors. According to [21], the partial safety factor for material characteristics is indicated with $\gamma_M$. Finally, $z$ is an arbitrary design variable such as the cross-sectional area or the section modulus. The reliability of the component is verified, when Equation 4.6 shows values larger than zero.

In the structural codes different partial safety factors are specified for different materials and for different types of loads. Furthermore, when more than one variable load is acting, load combination factors are multiplied on one or more of the variable loads. Therewith, codes take into account the likelihood of the variable loads’ simultaneous presence.

The partial safety factors together with the characteristic values are introduced in the design codes to ensure a certain minimum reliability level for the structural components. The principle is illustrated in Figure 4.4 for the simple case with two uncertain variables, namely, the resistance $R$ and the load $S$. Since material parameters of different material have different uncertainties associated to them, the partial safety factors of different materials are generally different.

![Design values, characteristic values and partial safety factors of an uncertain load and resistance.](image)

In accordance with a given design equation, such as e.g. Equation 4.6, a reliability analysis may be made by using a safety margin similar to the design equation, see Equation 4.7. However, here design values for the resistance and load variables are now replaced by basic random variables.

$$ g(X) = z R - (G + Q) $$

(4.7)

Considering Equation 4.7 together with probabilistic models for the basic random variables $R$, $G$ and $Q$, it is possible to determine the value of the design variable $z$ so that an maximum allowable failure probability is not exceeded. Such a design could be interpreted as being an optimal design because it exactly fulfils the given requirements to structural reliability.
Having determined the design variable $z$, one may also calculate the corresponding design point in the original space, i.e. $x_d$ for the basic random variables. This point may be interpreted as the most likely failure point, i.e. the most likely combination of the outcomes of the basic random variables leading to failure. Partial safety factors may be derived from the design point for the various resistance variables by

$$\gamma_X = \frac{x_k}{x_d} \quad (4.8)$$

and for load variables by

$$\gamma_X = \frac{x_d}{x_k} \quad (4.9)$$

Here $x_d$ is the design point of the considered variable $X$ and $x_k$ the corresponding characteristic value.

### 4.5 Rare events, vulnerability and robustness

In structural design codes the treatment of foreseeable hazards, such as extreme natural load events and accidental loads to structures, follows the same principles as for ordinary actions due to structural use and environmental loads. The design is verified by using a design equation of the same form as Equation 4.6. Hereby, of course the assignment of characteristic values must reflect the characteristics of the considered type of loading. Furthermore, the partial safety factors as well as load combination factors shall account for the usually high uncertainty associated with the intensity of accidental loads and the relative short duration of these loads.

It is a common basis in modern codes, such as e.g. the Eurocodes, that accidental loads, which are neither foreseeable nor prescribed otherwise, must be taken into account implicitly by certain requirements to the robustness of the structure. That is by anticipating a certain structural damage, such as the loss of a limited number of columns in a building structure, and by requiring that the structure as a whole does not fail as a consequence of the damage within a time sufficient to safeguard the occupants and/or the functions of the structures.

Whereas the classification of exceptional structures proposed in Section 4.2 is a general classification, more specific classifications have been proposed in some design codes. As an example, the Eurocodes introduce a classification of building structures in accordance with their consequences of failure. Within this framework the consequence class 3 buildings (high consequences) comprise:

1) Hotels, flats, apartments and other residential buildings exceeding 15 storeys.
2) Educational buildings exceeding 15 storeys.
3) Retailing premises exceeding 15 storeys.
4) Hospitals exceeding 3 storeys.
5) Office buildings exceeding 15 storeys.
6) All buildings to which members of the public are admitted in significant numbers exceeding 1'000 m$^2$ per storey.
7) Non-automatic car-parking exceeding 6 storeys.
8) Automatic car parking exceeding 15 storeys.
9) Leisure centres exceeding 2'000 m$^2$.
10) Stadium accommodating more than 5'000 spectators.

Furthermore, for this classification the Eurocodes suggest that risk analysis are carried out to document that the structures have a sufficient safety in regard to foreseeable as well as abnormal hazards. Such hazards should be considered specifically for the individual structures and in principle include all relevant natural as well as unintentional and malevolent human actions.
Chapter 4 – Structural reliability and vulnerability assessment

Risk analysis of structures subject to accidental loads can be approached by consideration of the three steps illustrated in Figure 4.5, see also [190].

\[
C_F = \sum_{i=1}^{N_H} \sum_{j=1}^{N_D} \sum_{k=1}^{N_P} C(P_i | D_j) \cdot P(D_j | H_i) \cdot P(H_i)
\]  

(4.10)

Furthermore, if it is assumed that a structure is subjected to \(N_H\) different hazards, it may be assumed that the hazards may damage the structure in \(N_D\) different ways, which can be dependent on the considered hazards. In addition, the performance of the damaged structure can be discretized into \(N_P\) adverse states with corresponding consequences \(C(P_i)\), the total failure consequences \(C_F\) can be assessed by Equation 4.10.

For a defined reference period, \(P(H_i)\) is the probability of occurrence of the \(i\)th hazard, \(P(D_j | H_i)\) is the conditional probability of the \(j\)th damage state of the structure given the \(i\)th hazard and \(P(P_k | D_j)\) is the conditional probability of the \(k\)th structural performance conditional on the \(j\)th damage state. In principle, \(P(P_k | D_j)\) as well as \(C(P_i)\) can be highly dependent of time, for instance when a fire and a subsequent evacuation is considered. Finally, the overall failure consequences \(C_F\) should be assessed and compared to acceptable risks accordingly or e.g. be used directly in the assessment of the life cycle benefit function Equation 3.4.

Equation 4.10 can form the basis for risk assessment of structures not only subject to accidental loads, but also for risk analysis of structures subject to ordinary loads. In this case, the hazards shall be interpreted as structural failures due to specific load combinations. Then the transition of the hazards over the damage states to the structural performance can be omitted.

To assess the risk according to Equation 4.10, necessitates that both the probabilities and the consequences are evaluated. Considering the assessment of consequences in terms of costs and fatalities, this issue is covered in Chapter 2 and Annex B. The assessment of the probability of occurrence of different hazards is not treated in general terms in the present context. Here it is noted that hazards and their occurrence probability are highly related to the considered type of structure, the geographical location of the structure as well as the use of the structure. For natural hazards useful information in regard to their occurrence probabilities can be gathered from public available sources, for instance the World Map of Natural Hazards developed by Munich Re [107]. For hazards due to fires and explosions information may be found in [86] and [62]. For hazards due to malevolent human actions the available statistical information is insufficient as basis for assessing the frequency.
of such events. For a specified building it might be possible to establish models for the occurrence probability of e.g. terrorism attacks as suggested in [97].

The probabilistic analysis of a given damage and a given hazard is not a trivial task and might be undertaken by a separate risk analysis in the principle form suggested in [190], where however, the main focus is directed to impact, subsequent fires and explosions. This approach follows closely the methodology, which has been successfully implemented in the petrochemical process industry during the last 1-2 decades, see also [199].

Assessing the probability of adverse events for the building in various damaged conditions can be readily undertaken by using the principles of structural reliability theory as outlined in Section 4.3. It should be noted that structural analysis for the performance assessment of damaged structures are significantly more demanding than those usually applied for the purpose of design verification.

Whereas, structural vulnerability can be assessed in terms of the expected extent of immediate damage (immediate consequences) following the occurrence of a hazard, the notion structural robustness refer to the ability of a structure to perform satisfactorily subject to a given state of damage. Here “satisfactorily” is often understood in the way that the overall performance of the structure is not reduced disproportionally to the extent of the damage.

The risk assessment steps indicated in Figure 4.5 point to the possible different strategies for risk control and reduction, if such are to be investigated for economical feasibility.

First of all, the risk might be reduced by reduction of the probability that the hazards occur, i.e. by reducing \( P(H) \). Considering e.g. ship impacts on bridge pier structures, the hazard (the event of a ship impact) can be mitigated by construction of artificial islands in front of the bridge piers. Similarly the risk of explosions in buildings might be reduced by removing explosive materials from the building. With regard to malevolently introduced hazards in buildings, one option might be to increase the security surveillance.

Secondly, the risk might be reduced by reducing the probability of significant damages for given hazards, i.e. \( P(D | H) \). Considering e.g. the damage which might follow as a consequence of the initiation of fires, risk reduction might be achieved by inactive and active fire control measures (e.g. fire protection of steel members and sprinkler systems).

Finally, the risk might be reduced by reducing the probability of adverse structural performance given structural damage, i.e. \( P(P | D) \). This might e.g. be undertaken by designing statically redundant structures allowing for alternative load transfer for the case when the static system was changed due to damages.

### 4.6 Modeling of system performance and structural damage

Generally, a given hazard might lead to a set with a large number of different damage states for a given structure. The possible damage states can, at best, only be characterized by means of their probability of occurrence. As outlined in the foregoing section, the assessment of damage states and their occurrence probabilities requires that detailed quantitative risk analysis is performed for a given structure. As the present study does not aim to consider any particular structure but to provide a general insight into the aspects of consequences and acceptance criteria for the design of extraordinary building structures, a simplified model for the characterization of structural damage is introduced in the following as well as for the modeling of the structural performance of the damaged structure. Even though the model is a gross simplification of the behavior of real structures subject to real damages, it does allow for an engineer to characterize damage states and to represent the resulting structural performance accordingly.

The basic principle behind the idealization is that it is assumed that the reliability of a structure in a certain condition (damaged and undamaged) is dominated by the reliability of a number of structural components.

In accordance with systems reliability analysis, the reliability of a redundant structural system can for instance be assessed by the probabilistic analysis of a series system of parallel systems. This is
Chapter 4 – Structural reliability and vulnerability assessment

illustrated in Figure 4.6, where four system failure modes (SFM) may lead to a system failure. Such a system failure mode are conditioned on the corresponding component failure modes (cfm).

<table>
<thead>
<tr>
<th>SFM1</th>
<th>SFM2</th>
<th>SFM3</th>
<th>SFM4</th>
</tr>
</thead>
<tbody>
<tr>
<td>cfm1,1</td>
<td>cfm2,1</td>
<td>cfm3,1</td>
<td>cfm4,1</td>
</tr>
<tr>
<td>cfm1,2</td>
<td>cfm2,2</td>
<td>cfm3,2</td>
<td>cfm4,2</td>
</tr>
<tr>
<td>cfm1,3</td>
<td>cfm2,3</td>
<td>cfm3,3</td>
<td>cfm4,3</td>
</tr>
<tr>
<td>cfm1,4</td>
<td>cfm2,4</td>
<td>cfm3,4</td>
<td>cfm4,4</td>
</tr>
<tr>
<td>cfm1,5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cfm1,6</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

cfm ... Component failure mode
SFM ... System failure mode

Figure 4.6 Series system of parallel systems modeling failure of redundant structural systems.

Each of the parallel systems represents different sequences of componential failure leading to structural failure. The characteristics of the individual components at failure may be either ductile, e.g. yield stress failure modes, or brittle e.g. instability failure modes. If an element is behaving ductile at failure, it still permits transfer of loads corresponding to the ultimate capacity; whereas if the component behaves brittle, no load transfer is possible after the component failure.

It is clear that systems reliability analyses are necessary in order to assess the performance of a given structural system. As structural systems are unique, some idealizations are thus required to facilitate a more general assessment of the performance of exceptional building structures. When serviceability failures are neglected and only ultimate limit states are considered, then the dominant failure modes may be assumed to be column bending and column compression due to wind, live and permanent loads. One possible idealization could be to consider system failure to take place as failure of a sequence of structural components, which transfer vertical loads from one storey to another.

When the components in the parallel system are fully correlated or when failure occurs as column instability, then an upper bound of the failure probability can be identified. In this case the system reliability is dominated by the component with the lowest reliability. When the components are not fully correlated and behave ductile at failure, then the system might be approximated by another system. This system consists of only the components with the lowest reliability and those, which at failure are utilized more or less equally. In many cases the identification of such systems can be performed approximately on a qualitative basis, thus avoiding rigorous and time consuming structural systems reliability analysis.

The quantitative reliability analysis of the approximate system with \( n \) components can be performed by evaluating the system’s reliability index \( \beta_s \) given by Thoft-Christensen and Baker [187].

\[
\beta_s = \beta_C \frac{n}{\sqrt{1 + \rho_{EQ} (n-1)}} \tag{4.11}
\]

where \( \beta_C \) is the smallest reliability index of the considered components and the equivalent coefficient of correlation \( \rho_{EQ} \) is given by

\[
\rho_{EQ} = \frac{1}{n(n-1)} \sum_{i 
eq j}^{n} \rho_{ij} \tag{4.12}
\]

In Equation 4.12 \( \rho_{ij} \) are the correlation coefficients between the system components.
The systems reliability index as evaluated from Equation 4.11 can then be evaluated provided that the reliability indexes of the relevant components and their correlation coefficients are available. This information may in turn be easily provided in terms of components reliability analysis with knowledge about the components failure modes and corresponding probabilistic models.

A damaged system can be analyzed exactly in the same manner as an undamaged system. However, modifications of the considered system’s idealization have to be considered. For the reliability analysis of a damaged system consideration must be given to the likelihood of simultaneously occurring extreme loads due to floor and wind loads together with the damage. This feature must be accounted for in the probabilistic modeling of the loads.

4.7 **Vulnerability to fire loads**

Steel structures heat up quickly in case of fire due to their large ratio of heated perimeter over cross-sectional area $A/V$ and the high thermal conductivity of steel. Because of elevated temperatures in case of fire, the strength and stiffness of steel decrease rapidly, see Figure 4.7. In order to prevent an early decrease of the strength and stiffness and to guarantee the required fire resistance, steel structures must usually be protected.

![Graph showing reduction factors $k_0$ for class 1 to 3 members according to [23] (left) and stress-strain relationship for steel S235 at elevated temperatures (right).](image)

Applying insulating fire protection material represents a common method for protecting steel members. A variety of insulating materials have been developed, including mineral-fiber or cementitious spray-applied materials, gypsum board, and concrete. Insulation may be sprayed directly onto the steel columns or beams. Alternatively, insulation boards may be used to form a protective box.

Simplified design methods for calculating the temperature increase of protected steel structures in fire were developed by Wickström [204] and Melinek and Thomas [102] and were included for example in [23]. The temperature increase $\Delta \theta_{at}$ of an insulated steel member during a time interval $\Delta t$ may be obtained from Equation 4.13 according to [23].

$$\Delta \theta_{at} = \frac{\lambda_p \cdot A_p}{d_p \cdot c_a \cdot \rho_a} \left( \theta_{g,t} - \theta_{a,t} \right) \Delta t - \left( e^{\phi \theta_0} - 1 \right) \Delta \theta_{g,t} + \phi = \frac{c_p \cdot \rho_p}{c_a \cdot \rho_a} \cdot d_p \cdot A_p / V$$

where $A_p / V$ is the section factor for steel members insulated by fire protection material, $A_p$ is the appropriate area of fire protection material per unit length of the member, $V$ is the volume of the
member per unit length, $c_a$ is the temperature dependent specific heat of steel, $c_p$ is the specific heat of the fire protection material, $d_p$ is the thickness of the fire protection material, $\theta_{a,t}$ is the steel temperature at time $t$, $\theta_{g,t}$ is the ambient gas temperature at time $t$, $\Delta \theta_{g,t}$ is the increase of the ambient gas temperature during the time interval $\Delta t$, $\lambda_p$ is the thermal conductivity of the fire protection system, $\rho_a$ is the unit mass of steel, and $\rho_p$ is the unit mass of the fire protection material.

Based on the time dependent temperature development in steel members, fire resistance can be verified by using the temperature dependent strength and stiffness properties of steel and well known design methods [23] or by using charts [61].

All design methods for calculating fire resistance of steel structures assume a completely protected member. However, in practice locally missing fire protection material is not unusual as can be seen in Figure 4.8, which shows steel beams with partially missing fire protection. The partial loss may occur due to impact, accidental damage, improper application or removal in the area of connections and installations. Therefore, the consequences of partial loss of fire protection are of special importance for establishing an optimal level of safety for exceptional building structures.

![Figure 4.8 Partial loss of fire protection at steel beams.](image)

Partial loss of fire protection material reduces the fire resistance of protected steel members markedly. Therefore, a better knowledge of the fire behavior of steel members with locally damaged fire protection is important to assess the reliability of fire protected steel structures. Predominantly the behavior of columns needs to be studied in detail, taking into account stability problems.

The results of a heat-transfer analysis of steel columns with partial loss of fire protection by using the finite element method were presented by [105]. A simplified approach in the temperature domain using the temperature endpoint criteria for steel columns included in [5] – an average temperature of 538°C and 649°C at a single point – was used to calculate the fire resistance of steel columns with partially missing fire protection. As expected, the study endorsed that only a small partial loss of fire protection strongly reduces the fire resistance.

[91] conducted tests to examine the fire resistance of steel columns with box-shaped sections subjected to fire with a temperature profile as specified in ISO 834 and hydrocarbon standard fire curve. In two of five experiments, 25% of protective material was taken off to simulate a situation with damaged fire protection. The experiments showed that the fire-resistance of columns is significantly reduced, even if only 25% of the protection material is missing. [91] concluded that a failure of columns could occur within 1/20 of its entire fire endurance period. However, the previously mentioned analysis in the temperature domain and the few temperature fields studied from numerical calculations and few column tests are not sufficient to fully understand and describe the stability behavior of protected steel columns with partial loss of fire protection material. Therefore, the fire behavior was studied by using a three-dimensional finite element heat transfer and
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structural model, by taking into account geometrical nonlinearities, local temperature distributions, thermal strains, and temperature dependent material properties. The model allows studying the effect of the size and location of missing protection on the fire resistance as well as the relationship between thickness of the fire protection material and fire resistance with parametric studies.

The results of the study confirm that local damage of fire protection is a decisive factor for the fire resistance of steel columns. Predominantly, for heavily protected columns the decrease of fire resistance time is very important, e.g. a small damaged area of 264 cm² on a IPE 240 section reduces the fire resistance of columns from approximately 90 min to less than 45 min. Therefore, it is important to avoid damaged areas by periodical repairs or by using robust fire protection systems.

4.7.1 Numerical model for fire resistance

For heat transfer analysis in fire, a number of specific numerical computer codes have been developed, e.g. [179], [82] and [69], based on finite elements or finite differences. Furthermore, many general purpose finite element packages can be adapted for heat transfer analysis in fire. For this study, the finite element program ABAQUS [1] was used. A main advantage of general purpose finite element programs is that they are also applicable for the mechanical analysis, e.g. bifurcation and thermal stress analysis.

4.7.1.1 Basic parameters

For the study, two different cross-sections were used, an IPE 240 and a HEB 300. The cross-sections were modeled by using the centre line dimensions. DS4 [1] shell elements for the heat transfer analysis and corresponding S4R [1] shell elements for thermal stress analysis were used. In order to reach an acceptable numerical calculation time as well as precise temperature and stress/strain results, a mesh refinement was modeled around the partial loss of the fire protection. For the buckling and thermal stress analysis, one rigid body condition was attached to each end of the column. The boundary conditions were assigned to the reference points of the rigid bodies. Deformation in the axial direction was possible, but twisting about the longitudinal-axis was prevented. Two horizontal restraints were applied at each reference point at the centroid of the cross-section.

4.7.1.2 Material properties

The temperature dependent thermal and mechanical properties for steel were taken from [23] for steel grade S 235. The strain-hardening option for steel was considered. Four different thicknesses of spray applied fire protections using three different thermal material behaviors were analyzed. The material properties used in the heat transfer analysis were taken from tests [40] and are presented in Table 4.1. For all coatings a specific heat of 1000 J/kg°C was assumed.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density [kg/m³]</th>
<th>Thermal Conductivity [W/m°C]</th>
</tr>
</thead>
</table>
| Coating I (13 mm) | 498 | 0.1 at 20°C  
0.1434 at 513°C  
0.1560 at 675°C  
0.1976 at 787°C  
0.22 at 1100°C |
| Coating II (19.5 mm) | 486 | 0.1 at 20°C  
0.1506 at 410°C  
0.2107 at 726°C  
0.22 at 1100°C |
| Coating III and IV (32.1 mm and 37.5 mm) | 486 | 0.08 at 20°C  
0.09 at 100°C  
0.11 at 300°C  
0.1275 at 447°C  
0.15 at 600°C  
0.2 at 1100°C |
4.7.1.3 Finite element analysis

In order to calculate the fire resistance period, finite-element analyses were performed under transient thermal and mechanical conditions. This implies that the load is raised up to the target level and then kept unchanged during the increase in temperature due to fire exposure.

Each analysis was carried out in four steps. The first step was an eigenvalue buckling analysis, in which the buckling modes were obtained and the deflection profile of the lowest buckling mode (buckling about the minor axis) was used to determine the initial column imperfections. Additional geometrical imperfections were used instead of structural imperfections. In the second step, the load-carrying capacity at ambient room temperature was determined by using the so called modified riks method (an arc length method). After increasing the applied load incrementally up to 60% of the load-carrying capacity (step three), the time dependent temperature values were applied, which were previously calculated in the heat-transfer analysis (fourth step).

For cross-sections exposed to fire on all sides, the thermal boundary condition in the heat transfer analysis may be described by a combination of radiant and convective heat flux. The heat flux from a unit surface area is given in Equation 4.14.

\[
q = \alpha_c \cdot \left( \theta - \theta^0 \right) + \varepsilon \cdot \sigma \cdot \left( \theta^2 - \theta^0 \right)^4 - \left( \theta^0 - \theta^z \right)^4
\]

where \( q \) is the heat flux to the unit surface area, \( \alpha_c \) is a convective heat transfer coefficient, approximately \( \alpha_c = 25 \text{ W/m}^2\text{C} \) on fire exposed sides [22], \( \theta \) is the temperature at this point on the surface, \( \theta^0 \) is a reference sink temperature value (Equation 4.19), \( \sigma \) is the Stefan-Boltzmann constant, \( \varepsilon \) is the resultant emissivity of the surface and the fire, approximately \( \varepsilon = 0.5 \) on the fire exposed sides, and \( \theta^z \) is the temperature of absolute zero on the scale being used (\( \theta^z = -273.15\text{°C} \)). For this study, the fire exposure \( \theta \) was taken according to the standard ISO 834 temperature time curve.

\[
\theta^0 = 20 + 345 \log_{10}(8t + 1)
\]

The thermal stress analyses were carried out by using a geometrical and physical nonlinear Newton-Raphson method.

4.7.2 Parametric study

The finite element method is a powerful tool to extend the range of a limited number of fire tests and to gain knowledge about the structural behavior in fire that is difficult to be established by testing. In a parametric study, the finite element program was used to get a general understanding of the thermal and mechanical behavior of insulated steel columns with partial loss of fire protection, including different cross-sections, different thicknesses of coating, and different sizes and locations of the missing fire protection.

Two different steel cross-sections were used to evaluate the effect of flange and web thickness: IPE 240 and HEB 300 sections. The column height for this analysis was 3'500 mm and the columns were modeled as simply supported at both ends. The utilization factor \( \mu_0 \) of all columns was 0.6. The missing protection was at the inner side of the initial bow imperfection, at half of the column length. The shape of the global imperfections was derived from the elastic buckling mode.

Simulations have been performed for coating thickness of 13 mm, 19.5 mm, 32.1 mm, and 37.5 mm. Table 4.2 shows the calculated fire resistances of the cross-sections without coating and with undamaged coating.

<table>
<thead>
<tr>
<th>Table 4.2</th>
<th>Fire resistance with undamaged fire protection material (utilization factor ( \mu_0=0.6 )).</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without coating</td>
</tr>
<tr>
<td>IPE 240</td>
<td>8.3 min</td>
</tr>
<tr>
<td>HEB 300</td>
<td>10.9 min</td>
</tr>
</tbody>
</table>
Vulnerability to fire loads

The decrease of fire resistance of steel columns as a function of partial damage of protection at the middle of the flange is shown in Figure 4.9. Simulations have been performed for columns whose fire protection was removed locally in an almost square shape with an area of 48 cm², 120 cm², and 264 cm². The curves of the IPE 240 cross-section level off for larger areas probably due to the change of the shape of the missing area from approximately a square to a rectangle.

Figure 4.10 shows the fire resistance of the steel columns as a function of the area of missing fire protection located at the middle of the web. As shown in Figures 4.9 and 4.10, the consequences of missing fire protection were more significant for thicker protection and longer fire resistance. The fire resistance of an IPE 240 column with coating IV (37.5 mm) decreased from 86 min without missing protection to 39 min for 264 cm² of missing protection, see Figure 4.9. A column with 2 hours fire resistance without missing protection (HEB 300 with coating IV) reached only 69 min fire
resistance if 264 cm$^2$ protection material was removed at the middle of the flange. A comparison
between Figures 4.9 and 4.10 shows that the influence of a missing protection at the web on the fire
resistance was less important than for missing protection at the flange. The influence of the location
of the removed fire protection at the flange can be seen in Table 4.3. The damaged area was either
located at the middle of the flange or at the edge of the flange at the inner side of the initial
imperfection. The size of the missing protection area was 48 cm$^2$.

<table>
<thead>
<tr>
<th>Coating II (19.5 mm)</th>
<th>Coating III (32.1 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage in the middle of the flange</td>
<td>Damage at the edge of the flange</td>
</tr>
<tr>
<td>IPE 240</td>
<td>36.1 min</td>
</tr>
<tr>
<td>HEB 300</td>
<td>56.1 min</td>
</tr>
</tbody>
</table>

Table 4.3 indicates that the influence of the location of the missing fire protection on the fire
resistance was small. Missing fire protection and consequently higher temperatures in the inner side
of initial imperfections with resulting compressive stresses affected the fire resistance slightly more
than on the outer side with resulting tensile stresses.

In addition to the decrease of the fire resistance time due to partial loss of protection material and the
loss of strength and stiffness, a supplementary decreasing influence for the load-carrying capacity of
steel columns at elevated temperatures was observed during the finite element analysis. Local
instability effects occur in the range of large failures of protection material.

4.7.3 Summary
Partial damage of fire protection resulting from impact, accidental damage, improper application or
removal in the area of connections and installations has a decisive influence on the fire resistance of
steel members. The previous chapters present a parametric study by investigating the influence of the
size and location of missing fire protection on the fire resistance time of steel columns by using the
finite element method, including buckling and thermal stresses.

Even if only small areas of fire protection are removed from the columns, a significant reduction in
fire resistance is observed. Expectedly, for columns with partial loss or damage of fire protection, the
thickness of the coating in the undamaged surface is of minor influence. In addition to global
buckling, in some cases also local buckling occurred near areas of damaged protection at low
geometrical plate slenderness due to elevated temperatures as shown in [64] and [65]. It is therefore
important to avoid damaged areas by periodical maintenance or to use robust protection systems,
such as steel-concrete composite structures.

4.8 Robustness to impact and fire damage
In the following, a simple model is used to investigate concepts to reduce the risk of progressive
collapse of buildings. Alternative structural systems are investigated using the World Trade Center
towers as a benchmark.

Accidental actions like earthquakes, explosions, fires or impacts can cause the failure of one or more
stories of a multistory building. When this happens, the upper part of the building starts falling down
acquiring a large amount of kinetic energy. If the lower part of the building is not able to withstand
this impact, the fall of the structure can not be stopped and a progressive collapse of the whole
structure follows. This collapse mechanism is also described in [201] and [202] as one of the two
possible reasons for progressive collapse. The other is when a collapse occurs on stories above the
initially collapsed story. As other authors have already pointed out, possible measures to prevent the
total collapse of a building basically consist of either designing or strengthening the structure, to
sustain the dynamic impact load, or to install a cushioning system, which is able to dissipate the
kinetic energy resulting from the free falling portion of the structure.
In the following, three alternative bearing systems and one cushioning system are designed to provide adequate impact resistance and structural safety to buildings of the same type as the World Trade Center towers. Their influence on the probability of failure of a story – which may result in the progressive collapse of the whole structure – is also taken into account.

4.8.1 Basic assumptions

4.8.1.1 Failure of a story

A story fails, when after the accidental action, its residual bearing capacity is lower than the vertical loads acting on that story. Only vertical load-bearing members like columns and walls are considered. Their load-bearing capacity is assumed to be equal either to their buckling resistance (columns) or their maximum compression strength (walls). As accidental action an aircraft impact followed by fire, similar to the load scenario of the WTC, is considered. The assumed damages caused by the impact are: 1) the total collapse of some structural elements of the structure and 2) the partial loss of fire protection of some other elements. The rest of the structure is assumed to be undamaged. To estimate the residual bearing capacity after fire, the maximal temperatures in the structural members are calculated, assuming the hydrocarbon fire curve according to [22], with fire duration of 120 minutes. This is a much more severe fire action than in the case of the WTC.

4.8.1.2 Progressive collapse

A simple model is assumed. The part of the structure below the failed story is modeled with an elastic spring, as shown in Figure 4.11. It is compressed by the upper part of the building, which is assumed to fall freely.

The impact is presumed to be completely elastic, implying a complete transformation of the kinetic energy into elastic deformation energy, which is stored in the spring. This model is also used in [10] and [11] for the investigation of the WTC’s failure. The model serves to estimate an equivalent static load \( F_{\text{dyn}} \), which describes the action of the dynamic impact on the lower part of the building. Considering a simple energy balance, this leads to an equation for the equivalent static load \( F_{\text{dyn}} \), where \( m \) is the mass, \( C \) the elastic spring stiffness of the lower part of the building and \( h \) the story height.

\[
F_{\text{dyn}} = mg + \sqrt{(mg)^2 + 2Cmgh} 
\]  

(4.16)

Figure 4.11  a) Failure of a story, b) spring model and c) final state after free fall of the mass \( m \) from a height \( h \) with a spring compression \( x \) [10], [11].

Depending on the considered scenario, the number of stories located below the airplane impact varies. This leads to different spring stiffness \( C \) of the lower part of the building. The total height of the building is constant.

4.8.2 Possible measures against progressive collapse

In the following, measures to prevent the progressive collapse of a building are briefly reviewed. They either reduce the impact load \( F_{\text{dyn}} \), increase the resistance of the structure, or dissipate energy. It
is clear that changes in these structural characteristics will change the structural response to other load cases. This effect is however not considered in the present context.

4.8.2.1 Reduction of impact load

A reduction of the stiffness $C$ leads to a reduction of the equivalent static load $F_{\text{dyn}}$. Therefore, a stiffness reduction without reduction of load bearing capacity would lead to better behavior of the structure. The stiffness for a centrically loaded truss is

$$C = \frac{EA}{l},$$

where $E$ is the elastic modulus, $A$ the cross-sectional area and $l$ the truss length. Materials with maximal yield strength to stiffness ratio should be preferred because for a given load, the required yield strength is inversely proportional to the area $A$. Therefore, for such an application steel is better suited than concrete. For example, a section made of A514 steel ($f_y = 690 \text{ MPa}$) has a three time smaller axial stiffness $EA$ than a section with the same strength made of C60/70 concrete ($f_{\text{cd}} = 40 \text{ MPa}$).

Another measure to decrease the impact load $F_{\text{dyn}}$ is to design a lightweight bearing structure. Equation 4.16 shows the strong dependence between $F_{\text{dyn}}$ and the mass of the falling part of the building. Also in this case steel is better suited than concrete because, assuming equal strength, a steel member is lighter.

4.8.2.2 Bearing structure designed for impact load

The structural system of the building should be designed or strengthened, so that its load-bearing capacity is not less than the equivalent static load $F_{\text{dyn}}$ given by Equation 4.16. It is noted that in this study the impact load is calculated assuming an elastic response of the structure, while the strength of the section is computed according to the theory of plasticity.

4.8.2.3 Cushioning system

The goal of a cushioning system is to dissipate the kinetic energy of the free falling part of the building. To this purpose, the contribution of steel columns is marginal. Bažant and Zhou have shown in [10] and [11] that the buckling of the steel columns of WTC dissipated only 1% of the total kinetic energy. Therefore, plastic cushioning provided by special energy-absorbing materials like aluminum foam might provide the needed energy dissipation. Such materials exhibit a great plastic strain capacity, up to 50%, and can be fitted into specially designed columns near the main load-carrying structure. To dissipate the kinetic energy, however, a part of the structure below the impact zone is destroyed.

4.8.3 Investigations using WTC as a benchmark

In the following section, the effectiveness of four different structural systems against progressive collapse are investigated assuming three different scenarios called Safety 10, 30 and 50, where 10, 30 and 50 indicate the number of falling stories. For example, in case of scenario Safety 30, the 80th story of the 110-story WTC Tower is assumed to fail, and the 30 upper stories are assumed to fall freely on the lower part of the building. In three of the four structural systems the structure was designed to ensure adequate safety against progressive collapse. They are able to carry the equivalent static impact load without permanent damage. The fourth structural system is provided with a cushioning system which is able to dissipate the kinetic energy resulting from the free fall of the upper stories of the building.

4.8.3.1 Description of the assumed structural systems

4.8.3.1.1 Actual state

Geometry and material properties of the WTC Twin Towers are summarized in [58] and, where needed, missing data are assumed by using engineering judgment. Each considered building has 110 stories of 3.66 m height, and a total weight of 370,000 t. The vertical structural system consists of 240 perimeter columns and 47 core columns. For the study it is assumed that the perimeter columns are made of A441 steel ($f_y = 345 \text{ N/mm}^2$) and have a 350x350 mm box-section, with plate thicknesses ranging between 6 and 75 mm. Core columns are made of A36 steel ($f_y = 248 \text{ N/mm}^2$) and have a box-
section of approximately 400x900 mm, with plate thicknesses ranging between 20 and 100 mm. The study considers a linear variation of the plate thickness over the height.

### 4.8.3.1.2 Steel structure

An alternative load-bearing structure is considered similar to the *Actual State* load-bearing structure. The only difference is the use of the high-strength A514 steel \( (f_y=690 \text{ N/mm}^2) \) for all columns and the increase of the plate thicknesses for both perimeter and core columns, in order to ensure the structural safety.

### 4.8.3.1.3 Composite structure

The core columns are covered with a 150 mm C60/70 concrete layer and the perimeter columns are filled with the same concrete. Grade A514 steel is used and increased plate thicknesses designed for all columns. Amount and location of the columns are the same as for the *Actual State*.

### 4.8.3.1.4 Concrete structure

Also for this scheme the perimeter columns are filled with C60/70 concrete and designed using high-strength A514 steel. The plate thicknesses vary, but are mostly smaller than in the *Actual State* case. Massive reinforced concrete walls of 1 m thickness replace core columns. The total length of these walls is designed to ensure a sufficient structural safety.

### 4.8.3.1.5 “Cushioning” structure

For this scheme, which is proposed by Newland in [112], the structural system remains the same as in the *Actual State*. In addition, a dissipating structure is installed close to the perimeter columns, consisting of cushioning columns made of aluminum foam. The cushioning structure is designed to slow down the falling part of the building, by plastic deformation of the energy-absorbing material over an impact zone of two stories (7.32 m). It is assumed that over this height the structure will be totally destroyed.

### 4.8.3.2 Effects of accidental actions

#### 4.8.3.2.1 Initial damage from aircraft impact

The damage to the structure, which is assumed for the *Steel Structure* during the Safety 10 scenario, is the same as the best estimate of the damage caused by the airplane impacts on the WTC Towers, i.e. 13% of the perimeter and 27% of the core columns [58]. The portion of undamaged elements is estimated at 10% for all four considered structures. The remaining elements suffer a partial loss of fire protection. This is a conservative assumption but more detailed data was not available. The amount of destruction sustained by the other three considered structures is estimated using an energy approach by assuming that the total deformation energy \( U \) dissipated by the collapsed elements (columns and walls) is the same for each structure. The final percentages of structural damage are listed in Table 4.4.

![Figure 4.12 Deformation energy.](image)

<table>
<thead>
<tr>
<th>Safety 10</th>
<th>Safety 30</th>
<th>Safety 50</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>PC</td>
<td>CC</td>
</tr>
<tr>
<td>Destroyed</td>
<td>13</td>
<td>27</td>
</tr>
<tr>
<td>Loss of FP</td>
<td>77</td>
<td>63</td>
</tr>
<tr>
<td>Undamaged</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 4.4 Damage due to airplane impact (FP=fire protection, PC=perimeter columns, CC=core columns and W=walls).
4.8.3.2.2 Residual bearing capacity after fire
Assuming a hydrocarbon fire exposure curve according to [22], a fire duration of 120 minutes and a fire protection consisting of either a 25 mm mineral fiber layer ($\lambda_p=0.116$ W/mK) [58] or a 150 mm concrete layer, the temperatures in the structural elements are calculated, see also Section 4.7. Afterwards, the residual load carrying capacity is calculated considering the associated loss of strength and stiffness according to [66], [63], [19] and [20]. The calculated residual load bearing capacity is shown in Table 4.5.

Table 4.5  Residual load bearing capacity (RBC) after fire (PC=perimeter columns, CC=core columns and W=walls).

<table>
<thead>
<tr>
<th>RBC [%]</th>
<th>Steel PC</th>
<th>Steel CC</th>
<th>Compos. PC</th>
<th>Compos. CC</th>
<th>Concr. PC</th>
<th>Concr. CC</th>
<th>Concr. W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Destroyed</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Loss of FP</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>17</td>
<td>6</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Undamaged</td>
<td>73</td>
<td>74</td>
<td>67</td>
<td>70</td>
<td>67</td>
<td>85</td>
<td></td>
</tr>
</tbody>
</table>

4.8.3.2.3 Total residual load bearing capacity
The total residual load bearing capacity after airplane impact and fire is summarized in Table 4.6. As a check of the proposed methodology, a similar calculation is done assuming an airplane impact on the 90th floor of the Actual State structure. Such a computation corresponds to a simulation of the WTC1 impact and the result that the structure is able to withstand the impact but collapses after the fire is in good agreement with the observed behavior of WTC1.

Table 4.6  Total residual load bearing capacity (RBC) (PC=perimeter columns, CC=core columns and W=walls).

<table>
<thead>
<tr>
<th>Total RLBC [%]</th>
<th>Steel PC</th>
<th>Steel CC</th>
<th>Compos. PC</th>
<th>Compos. CC</th>
<th>Concr. PC</th>
<th>Concr. CC</th>
<th>Concr. W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total RLBC</td>
<td>12</td>
<td>11</td>
<td>19</td>
<td>11</td>
<td>67</td>
<td>12</td>
<td>12</td>
</tr>
</tbody>
</table>

4.8.3.3 Results
When an impact occurs two outcomes are possible: 1) the story fails completely or 2) the story does not fail because the structure is redundant and has enough residual strength left [185]. Both outcomes are considered during the evaluation of the performance of the four considered structural systems. For both cases, the evaluation criteria are: additional weight, total loss of floor space, fire resistance and not least investment costs. The comparisons refer to the Actual State.

4.8.3.3.1 Story failure occurs
The different systems are designed to prevent a progressive collapse after the failure of a story. The main results are listed in Table 4.7.

Table 4.7  Main characteristics of the structural systems designed to resist the failure of a story (AW=additional weight, LFS=total loss of floor space and IC=investment costs).

<table>
<thead>
<tr>
<th>Design to withstand progr. collapse</th>
<th>AW $10^3$[t]</th>
<th>LFS $10^3$[m²]</th>
<th>IC $10^3$[tss]</th>
<th>AW $10^3$[t]</th>
<th>LFS $10^3$[m²]</th>
<th>IC $10^3$[tss]</th>
<th>AW $10^3$[t]</th>
<th>LFS $10^3$[m²]</th>
<th>IC $10^3$[tss]</th>
<th>AW $10^3$[t]</th>
<th>LFS $10^3$[m²]</th>
<th>IC $10^3$[tss]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety 10</td>
<td>7.2</td>
<td>244</td>
<td>20</td>
<td>48.2</td>
<td>2932</td>
<td>76</td>
<td>289.1</td>
<td>31389</td>
<td>705</td>
<td>0.6</td>
<td>130</td>
<td>10</td>
</tr>
<tr>
<td>Safety 30</td>
<td>58.3</td>
<td>791</td>
<td>79</td>
<td>102.4</td>
<td>3685</td>
<td>144</td>
<td>732.2</td>
<td>79861</td>
<td>1816</td>
<td>3.9</td>
<td>1075</td>
<td>71</td>
</tr>
<tr>
<td>Safety 50</td>
<td>106.1</td>
<td>1232</td>
<td>133</td>
<td>158.3</td>
<td>5540</td>
<td>227</td>
<td>1191.0</td>
<td>129956</td>
<td>2966</td>
<td>10.3</td>
<td>2807</td>
<td>185</td>
</tr>
</tbody>
</table>

Due to the low specific weight of aluminum foam, the Cushioning Structure is the lightest solution. The additional weight of the Concrete Structure is significantly larger because of the unfavorable yield strength/stiffness and yield strength/mass ratios of concrete. The Steel Structure is the best alternative in terms of loss of floor space. Only for scenario Safety 10 the Cushioning Structure yields the minimum loss of floor space because the cushioning columns are implemented only in the
12 upper stories while the increased sectional dimensions of the Steel Structure have to be provided along the entire height of the building.

The additional costs for safety are calculated taking into account material costs, assembling costs and loss of income due to reduced floor space. The price of high strength steel is assumed to be 800 USD/t and an improvement of the steel grade is estimated to cost 120 USD/t (from A36 and A441 to A514). The price of C60/70 concrete is 280 USD/m$^3$ while an aluminum foam price of 12'000 USD/m$^3$ is considered [112]. The cost of the loss of floor space is estimated at 640 USD/m$^2$ per year with a 3% interest rate. The design lifetime is considered to be 200 years.

For scenarios Safety 10 and Safety 30, the cheapest solution is ensured by the Cushioning Structure with 10 and 71 million USD, this again because the cushioning columns are inserted only in the 12 respectively 32 upper stories of the building. For scenario Safety 50, the Steel Structure yields the minimum costs because, compared to the other structures, the loss of floor space is significantly lower. In comparison, the cost of the two WTC towers (construction period 1966-1977) was 900 million USD [169]. Today they would cost 4.7 billion USD, or 2.35 billion USD per tower, see Annex B.

Major benefits of the concrete walls and partially of the composite elements are a higher fire resistance and a higher resistance against initial damage due to accidental action (see Table 4.4). Table 4.8 shows the minimum undamaged portion of a structure, designed to withstand the failure of a story, required to prevent the failure of a story. It is seen that all structures are very redundant and that the failure of a story is quite unlikely. Therefore, all structures, but the Cushioning one, are redesigned so that they are able to prevent story failure in the case the damage assumed in Table 4.4 and 4.5 occurs. This is considered in the following section.

Table 4.8 Minimum required percentage of undamaged structural element to prevent the failure of a story.

<table>
<thead>
<tr>
<th>Undamaged</th>
<th>Steel</th>
<th>Compos.</th>
<th>Concr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety 10</td>
<td>4</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Safety 30</td>
<td>6</td>
<td>7</td>
<td>9</td>
</tr>
<tr>
<td>Safety 50</td>
<td>7</td>
<td>8</td>
<td>11</td>
</tr>
</tbody>
</table>

4.8.3.3.2 Story failure doesn't occur

In the following, the progressive collapse is no longer considered because the failure of a story is prevented by designing all structural elements accordingly. The results are summarized in Table 4.9 and, as expected, they show significantly lighter structures compared to Table 4.7. The Steel Structure is clearly the cheapest for all scenarios, this again due to the small loss of floor space.

Table 4.9 Main characteristics of the structural systems designed to prevent the failure of a story (AW=additional weight, LFS=total loss of floor space and IC=investment costs).

<table>
<thead>
<tr>
<th>Design to withstand impact &amp; fire</th>
<th>Steel</th>
<th>Composite</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety 10</td>
<td>2.0</td>
<td>34</td>
<td>11</td>
</tr>
<tr>
<td>Safety 30</td>
<td>10.8</td>
<td>185</td>
<td>22</td>
</tr>
<tr>
<td>Safety 50</td>
<td>22.6</td>
<td>387</td>
<td>37</td>
</tr>
</tbody>
</table>

Certainly, the structural system required to prevent the failure of a story depends on the assumed initial damage. In this case the design to prevent the failure of a story is significantly more cost-effective compared to the design to withstand the failure of a story; even if in the latter case the structure is provided with cushioning elements.

4.8.4 Summary

The presented study considers the issue of how could buildings be designed to prevent progressive collapse following an accidental action. Therefore, two basically different approaches are considered: 1) a maximum damage is assumed and the residual structure is designed to carry the total loads...
preventing the failure of a story. 2) The failure of a story is assumed and the structure underneath is designed either to carry the relevant impact load or to dissipate the resulting kinetic energy.

Both approaches are investigated by using the WTC Towers as a benchmark. As in the real case, the considered accidental loading consists in an airplane impact followed by a fire. The results show that the first approach is the most cost-effective. Certainly the required structural capacity depends on the assumed damage. Should a more severe damage occur, a story would fail and it would be impossible to prevent a total progressive collapse of the building. Therefore, assumptions about initial damage should be supported by an extensive risk analysis. If alternative 2) is preferred and a story failure is accepted, the resulting structure is very robust and redundant but the resulting costs are almost prohibitive. Among the proposed solutions, the Cushioning Structure and Steel Structure are the best in terms of cost-effectiveness. It is worth noting that the cushioning system could be installed with relative ease in both new and existing structures.

For the considered case steel structures are more favorable than reinforced concrete structures. Due to their higher yield strength/mass ratio they may be designed for a lower impact load. Furthermore, steel is more efficient than concrete in the present context due to the higher yield strength (as an example, the yield strength of A514 steel is seventeen times higher than the compressive strength of C60/70 concrete). In addition the loss of very expensive floor space is less when compared to reinforced concrete structures. The major advantages of reinforced concrete elements are a better fire resistance and a higher resistance against initial impact damage. For instance, in the structural systems considered in this study, the total residual carrying capacity of reinforced concrete walls is about six times bigger than the one of the steel columns. Despite these advantages, the investigations show that both the Concrete and the Composite Structures are not very cost-effective due to their high area demand.
5 Acceptance criteria and optimality for the design of extraordinary building structures

In the following, a principle study illustrates the approach to optimal design as outlined in the foregoing. The study takes basis in the theoretical framework for optimal design, which is outlined in Chapter 3. Moreover, it makes use of the developed model framework for consequence modeling described in Chapter 2. The consequence models are utilized together with data on failure consequences of the failure of the WTC Twin Towers analyzed and described in Chapter 2 and Annex B. Finally, the analysis of the reliability of extraordinary building structures is pursued according to Chapter 4.

First, the problem of optimal design of extraordinary building structures in regard to ordinary load cases is investigated and the effect of different assumptions on the modeling of the interest rate is assessed. Thereafter, the problem of optimal design for fire load cases is addressed and various fire risk reduction measures are investigated in this context.

5.1 Design for ordinary load cases

The present section considers the optimal design of a high-rise building. For this purpose the effect of different designs of a short steel column on the expected life cycle costs is investigated for a high-rise building. A core column is considered subject to permanent and life loads. To start with, it is assumed that during the design lifetime of 200 years the risk due to fire may be neglected and the corresponding optimal design is identified. Thereafter, the effect of fire risk reduction measures is considered.

Failure events, which occur due to the effect of excessive permanent and/or live loads, can be described by the safety margin given by Equation 5.1.

\[
g(X) = z f_y \xi = \frac{a_G}{a_G + [1 - a_G]} \]  

Here, \( z \) is a design variable, e.g. the cross-sectional area, \( \xi \) represents model uncertainties and \( f_y \) is the yield strength. The product of these variables is the uncertain resistance of the considered column. The loading is composed of a permanent load \( G \) and a live load \( L \). \( a_G \) is the ratio between the load effect due to permanent load and the load effect due to the total load. In the present example, it is assumed to \( a_G = 0.4 \). The similarity between Equation 5.1 and Equation 4.6 indicates that using this approach, structural codes may be calibrated. However, this is not pursued in the following. The probabilistic models for the random variables are given in Table 5.1 closely following the JCSS Probabilistic Model Code [86].

For the evaluation of the life cycle costs it is assumed that if a structure fails a decision will be made to reconstruct it. The underlying decision process is illustrated in Figure 3.2b, whereas the assumption is also supported by experiences gained from the World Trade Center failure, see Section 3.2.1.1.

Applying the selected reconstruction strategy, the expected revenue becomes independent of the structural design; therefore, the minimization of the expected life cycle costs is equivalent to the maximization of the expected life cycle benefit, see Section 3.1. Hence, only the expected life cycle costs are evaluated.
Furthermore, it is assumed that the considered column is not subject to deterioration processes such as fatigue or corrosion. This allows to neglect inspections and repairs, see Section 3.2. Moreover, this permits to model the resistance as being constant over time and when the load is stationary as well, then it is suitable to assume that failures occur as realizations of a stationary Poisson process. Based on these assumptions, the expected life cycle benefit may be formulated analytically, see e.g. [168], [161] and [92].

Table 5.1  Stochastic Model.

<table>
<thead>
<tr>
<th>X</th>
<th>Dist. Type</th>
<th>E[X]</th>
<th>V[X]</th>
</tr>
</thead>
<tbody>
<tr>
<td>fr</td>
<td>LN</td>
<td>1.00</td>
<td>5%</td>
</tr>
<tr>
<td>ξ</td>
<td>LN</td>
<td>1.00</td>
<td>10%</td>
</tr>
<tr>
<td>L</td>
<td>G</td>
<td>1.00</td>
<td>40%</td>
</tr>
<tr>
<td>Lf</td>
<td>LN</td>
<td>1.00</td>
<td>15%</td>
</tr>
<tr>
<td>q</td>
<td>G</td>
<td>0.57</td>
<td>59%</td>
</tr>
<tr>
<td>qG</td>
<td></td>
<td>400 MJ/m²</td>
<td>25%</td>
</tr>
</tbody>
</table>

In order to evaluate the expected life cycle costs, all costs are normalized to the initial construction costs. The cost model is based on engineering experience and on the consequence models described in Chapter 2 and Annex B. From engineering experience and data from high-rise building projects, the costs of the steel structure (the main static system) are set to 18% of the total costs. Furthermore, it is considered that the costs for the steel structure vary linearly with the used steel.

For the case when the structure fails completely, two situations are considered. The first situation considers only the direct consequences as costs of failure. Direct costs includes economical consequences due to the lost structure, inventory, as well as rescue and clean-up efforts. Therewith, direct consequences are obtained, which are 2.5 times the reconstruction costs. The second situation considers follow-up consequences as suggested in Section 3.3. The follow-up consequences includes in addition to direct consequences the loss and damage of surrounding buildings and infrastructural facilities, consequences due to fatalities and the impact to the economy. In this situation the failure costs are estimated to lie in the interval between 7.5 to almost 20 times the reconstruction costs. In the following, the larger value is considered. Using these data it is implicitly assumed that the same multiplier effect and k-factor apply, as observed for the case of the WTC failure, see Annex C.2 and Annex B.4.

The life cycle costs are evaluated for two different rates of interest. The first interest rate uses as a basis the GDP per capita annual growth rate from [189] and derives a sustainable interest rate, see Section 3.4. This value is taken to be \( r = 3.5\% \). Secondly, a real interest rate of \( r = 6.0\% \) is used, which represents the preferences of private investors.

In the following, the expected life cycle costs are evaluated for \( \varphi \), the relative change of the cross-sectional area \( A \).

\[
\varphi = \frac{A}{A_0} - 1 \tag{5.2}
\]

Here, \( A_0 \) is the cross-sectional area for \( \varphi = 0 \), which represents a design with an annual probability of failure of \( 10^{-4} \). The annual probability of failure of \( 10^{-4} \) has been chosen from Table 5.2 by assuming that the structure can be classified as having large consequences in case of failure and having large relative costs of safety measures.

Table 5.2  Target reliability indexes and target failure probabilities for a one year reference period according to [86].

<table>
<thead>
<tr>
<th>Relative cost of safety measure</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major</td>
<td>Minor consequences of failure</td>
<td>Moderate consequences of failure</td>
<td>Large consequences of failure</td>
<td></td>
</tr>
<tr>
<td>Large (A)</td>
<td>( \beta = 3.1 ) (( p_F = 10^{-3} ))</td>
<td>( \beta = 3.3 ) (( p_F = 5 \times 10^{-4} ))</td>
<td>( \beta = 3.7 ) (( p_F = 10^{-3} ))</td>
<td></td>
</tr>
<tr>
<td>Normal (B)</td>
<td>( \beta = 3.7 ) (( p_F = 10^{-4} ))</td>
<td>( \beta = 4.2 ) (( p_F = 5 \times 10^{-5} ))</td>
<td>( \beta = 4.4 ) (( p_F = 10^{-4} ))</td>
<td></td>
</tr>
<tr>
<td>Small (C)</td>
<td>( \beta = 4.2 ) (( p_F = 10^{-3} ))</td>
<td>( \beta = 4.4 ) (( p_F = 5 \times 10^{-5} ))</td>
<td>( \beta = 4.7 ) (( p_F = 10^{-4} ))</td>
<td></td>
</tr>
</tbody>
</table>
5.2 Design for fire load cases

In continuation of the preceding study, the cost efficiency of fire risk reduction measures is investigated in the following. Passive fireproofing in terms of sprayed on concrete as well as active fire protection in terms of sprinklers are considered. Three combinations of fire risk reduction alternatives are analyzed, namely 1) fireproofing and sprinklers, 2) no fireproofing but sprinklers and 3) fireproofing without sprinklers. The safety margin for the situation when the reliability of the column is assessed conditional on a flashover is given as:

\[ g_{\beta}(X) = z f_{\beta} k_Y \xi_{\beta} - \left( \alpha_G G + [1 - \alpha_G] L_{\beta} \right). \]  

(5.3)

In Equation 5.3, \( \xi_{\beta} \) represents a resistance model uncertainty for the fire design situation, \( k_Y \) represents the reduction of the yield strength of the steel due to the increased temperature and \( L_{\beta} \) is the live load, which is representative in the case of a fire. According to [86], live loads comprise of sustained loads and intermittent loads, whereby sustained loads are modeled by rectangular wave processes and intermittent loads modeled by Poisson pulse processes. In case of a fire, there may be concentrations of people at some locations, e.g. staircases; however, at locations where there are no concentrations, the intermittent loads can be neglected and only sustained loads are acting on the structure. The result is a lower mean value of the life load in case of a fire. This is here found to be
equal to 57% of the mean value of the live load when no fire is considered. However the coefficient of variation increases to 59%. The probabilistic models are given in Table 5.1.

The steel strength reduction factor $y_k$ is evaluated as a function of the steel temperature. The air temperature is modeled by using the parametric temperature curve proposed in [22] and [171].

$$\Theta_g = 20 + 1325 \left( 1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*} \right)$$  \hspace{1cm} (5.4)

Where $t^*$ is given by

$$t^* = t \Gamma$$  \hspace{1cm} (5.5)

and $\Gamma$ by

$$\Gamma = \left( \frac{O}{b} \right)^2 \left/ \left( 0.04 m^{0.5} / 1160 J/m^2 s^{0.5} K \right)^2 \right. .$$  \hspace{1cm} (5.6)

With the utilized parameters $O = 0.04 m^{0.5}$ and $b = 1160 J/m^2 s^{0.5} K$ for the opening factor and the factor of the walls’ thermal conductivity, respectively, the parametric temperature curve is almost identical to the ISO temperature curve. In [18] it is also determined at what time the air temperature reaches its maximum at $t_{max}$ and at what rate it declines thereafter.

According to [20], the steel temperature can be obtained by numerical integration for both the cases with and without fire proofing, see also Section 4.7. [20] also provides the relation between $k_y$ and the steel temperature. Protected and an unprotected 900x400 mm rectangular hollow core steel columns with plate thicknesses of 35 mm are considered. The considered fire proofing consists of a 25 mm thick “sprayed-on” fireproofing material containing mineral fiber with a thermal conductivity of $\lambda_p = 0.116 W/(m K)$. 

According to [86], fire is assumed to occur at a rate of $10^{-6}$ per annum and square meter. Assuming further an area of $A = 400'000 m^2$, this amount to an average of 0.4 fires per annum. When no sprinklers are installed, the conditional probability of a flashover given an ignited fire is 12.7% and 0.5% when sprinklers are in place [62]. Furthermore, for the evaluation of the failure probability due to fire and life load, it seems appropriate to assume that these loads are independent.

The costs of sprinklers are assumed to be equal to 1.5% of the construction cost and the fireproofing costs are assumed to be equal to 5% of the construction costs. Costs due to a flashover, which does not lead to a structural failure, are assumed be equal to 5% of the construction costs, when there are no sprinklers. When sprinklers are in place, they will reduce damages significantly and in this situation the costs are assumed to be 10 times lower.

![Effect of different fire safety measures on the expected life cycle costs.](image)

Figure 5.2  Effect of different fire safety measures on the expected life cycle costs.
Figure 5.2 illustrates the expected life cycle costs of the three combinations of safety measures. It is seen that for the considered case the least expected costs are obtained when the structure is fire protected, has sprinklers and the structure is designed with $\varphi=11\%$. A structure without fireproofing but with sprinklers yields generally higher expected life cycle costs. Its minimum is achieved with $\varphi=21\%$. $\varphi=14\%$ minimizes the life cycle costs for a fire protected structure without sprinklers; nonetheless, this combination is less efficient than combination 1) and 2). Considering a structure without fire proofing and without sprinklers the expected life cycle costs are even higher. These costs are so high that they do not fit into the considered ranges of Figure 5.2.

Design for fire load cases

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6 Conclusions and suggestions for further research

An assessment of the consequences following the failure of the WTC Twin Towers on September 11, 2001 has been carried out. The assessment has facilitated the understanding of the complex interconnectivity of consequences of the incidents of the considered type and has formed the basis for operational models for consequence assessment for future purposes, such as design optimization.

The decision problem of design optimization has been formulated within the framework of the Bayesian decision analysis, utilizing and extending theoretical formulations developed in the most recent years. The framework facilitates the inclusion of follow-up consequences which in the light of the consequences experienced from the failure of the WTC Twin Towers play a predominant role.

By utilizing a three phase representation of the process leading from a load event to a potential failure, a methodical framework for risk assessment and design verification of structures in regard to usual as well as extraordinary load situations has been developed. The developed framework allows for a quantification of structural exposure, structural vulnerability and structural robustness and the way these factors influence the risk of failure.

Furthermore, preliminary investigations concerning the optimal design of extraordinary structures have been performed by utilizing the developed model framework. These investigations clearly show that an increased structural reliability is required when the design of structures similar to the WTC Twin Towers is considered. Finally, the studies illustrate how risk reducing measures, such as active and passive fire protection may be consistently quantified and taken into account in the design optimization problem.

Finally, with reference to conditions relevant for the WTC Twin Towers some principal studies are undertaken in regard to the vulnerability of passive fire protection as well as the possibilities to achieve sufficient robustness to avoid the development of a progressive collapse failure mode. These studies underline that for the case considered the passive fire protection is highly vulnerable and furthermore that sufficient robustness against progressive collapse is achievable but most likely not cost efficient.

During the project many aspects of optimal structural design have been considered with special focus on extraordinary building structures. Even though a consistent and operational methodology has been established for the purpose of optimal design and assessment of acceptance criteria, it is also clear that much more work and research is still needed.

The aspects concerning detailed modeling of the structural reliability of complex building structures subject to both ordinary and extraordinary load conditions is of high relevance. This would lead to a highly needed increased insight into the problem of establishing less vulnerable and more robust structures for different relevant load events.

More detailed structural optimization studies are required in order to consider all relevant ordinary and extraordinary load conditions. Such studies are needed in order to bring the present model framework into a stage where it can be utilized in the context of design code development.

Follow-up consequences play a predominant role in the overall consequence assessment. This insight has been strongly underlined by the present studies, however, points to the need of similar considerations in connection with other types of structures and other load events. One pressing case
would be to consider in detail the follow-up consequences which would follow from the effect of e.g. earthquake events acting on highly industrialized regions and financial centers. It is envisaged that also in such cases the consequences going beyond the direct loss of a number of buildings would imply that also more ordinary buildings should be designed to a higher reliability than presently incorporated into the design codes.

Finally, the issue of achieving optimal design decisions and at the same time sustainability has been considered; preliminary investigations have shown that the effect of the requirement to sustainability leads to different optimal designs. It should be underlined that the performed studies are very preliminary; however, it is believed that the proposed decision theoretical framework for the achievement of sustainability has a very high potential in the general context of sustainable decision making. So far, no other consistent theoretical formulation has been presented for this purpose. The proposed framework deserves to be further developed, adapted and tested on different relevant decision situations.
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Chapter 7 – References


A The World Trade Center

For the purpose of easy reference, the present chapter gives a short introduction and a historical review of the World Trade Center. The text mainly follows the building performance study [58], however, information from many other sources such as [184], [167], [74], [169] are also considered.

A.1 Location of the World Trade Center

The World Trade Center was located in Lower Manhattan in the Financial District of New York City in the United States of America. It comprised a 16 acre (64'750 m²) site bounded by Vessey Street in the north, Church Street in the east, Liberty Street in the south and West Street in the west. Between 1966 and 1985 the seven World Trade Center buildings WTC1 to WTC7 were built, whereas WTC7 was located just across Vessey Street. This site also accommodates stations for the Port Authority Trans Hudson (PATH) trains and the Metropolitan Transit Authority (MTA) subway, see Figure A.1.

A.2 Historical review

In January 1960, the development of a World Trade Center was recommended by the Downtown-Lower Manhattan Association, which was headed by David Rockefeller. In March 1961, this association issued a report, which recommended the construction of a World Trade Center and suggested that the Center could be built by the Port Authority of New York and New Jersey. In the following, the Port Authority of New York and New Jersey will be referred to as Port Authority. In January 1964, the Port Authority unveiled the architectural plans of a World Trade Center, which feature two 110-story towers. Finally, the construction began by breaking ground on August 5, 1966. It was followed by the erection of steel in August 1968. On October 19, 1970 the World Trade Center North Tower exceeded the height of the Empire State Building and became the world’s tallest building. Until that date, the Empire State Building held the record for forty years. In 1974, the World Trade Center passed the title on to the Sears Tower in Chicago. In mid December 1970, the first tenant moved into the North Tower WTC1 and a week later on December 23, 1970 the “topping-out” ceremony of the North Tower was celebrated; on July 19, 1971, the “topping-out” ceremony for the South Tower was committed. Finally, on April 4, 1973 the WTC was inaugurated by the Governors of New Jersey and New York William T. Cahill and Nelson A. Rockefeller.

By 1985, the remaining buildings WTC3 to WTC7 were built. WTC3 was a 22-storey hotel, whereas WTC4 and WTC5 were 9-storey office buildings, WTC6 was a 8-storey office building and WTC7 was a 47-storey office building. The seven buildings together provided approximately 12 million square feet (1.11 million m²) rentable floor space, which was occupied by various government and commercial tenants. Most of the tenants were from the financial and insurance industries.

During the lifetime of the World Trade Center, the New York Fire Department was called hundreds of times to the site. The firemen extinguished minor fires or arrived due to false alarms. However, the firemen were also called for major fires such as the one that occurred on February 13, 1975. The rescue teams also reacted to the bomb attack on February 26, 1993.

In July 2001, six weeks before the collapse of the WTC, the two towers were bought by a group led by Larry Silverstein Properties Incorporated. The building was transferred on a 99 year lease basis with a net present value of 3.24 billion USD.
A.3 Construction of the World Trade Center

A.3.1 Foundation

Due to the groundwater level and the nearby Hudson River, a ‘bathtub’ was built for the foundation of the WTC complex, underneath the west part of the WTC plaza, see [184]. This structure is 980 foot (299 m) long and 520 foot (158 m) wide; at the time of construction it was only 200 feet (61 m) away from the Hudson River. For the design of the bathtub’s perimeter wall, several boundary conditions had to be considered. Firstly, the operation of the PATH trains should not be disturbed; secondly, the support of the MTA subway tunnel (line 1 and 9) had to be considered because it is located at the bathtub’s east wall. Furthermore, the entry points of the PATH tube had to be reviewed, and the foundation of WTC1 and WTC2 on the bedrock within the excavation had to be considered as well as the geological situation. In the west of the bathtub, the schist bedrock can be found at a depth of 65 to 80 feet (20-24 m) and in the east at a depth of 55 to 75 feet (17-23 m).
For the construction of the bathtub, the slurry wall technique was used. It was the second time that slurry walls were used in the United States. In order to reach the bedrock, the slurry walls passed through fill cover, organic marine clay, glacial outwash sand and silt and glacial till and decomposed rock. The bathtub was formed by a 3,500-foot (1,067 m) long and 3-foot (91 cm) thick perimeter wall, which was set into the bedrock at depths as much as 80 feet (24 m). The slurry wall panels were 22 feet (6.7 m) wide and were temporarily supported by high-strength tendon tieback anchors. These tiebacks were installed through sleeves in the slurry wall after the excavation reached the corresponding level and the tiebacks were drilled into bedrock. In order to accommodate the excavated soil, a landfill across West Street was created. Today it is the site for Battery Park City (more than a million cubic yards (7.6 million m³)). After the bathtub's completion, the basement floors were constructed. They also served for the lateral support of the slurry wall; the temporary tiebacks were decommissioned and the sleeves were sealed. The subterranean structure consisted of 6 levels B1 to B6. These levels contained a shopping mall, mechanical and electrical building services, a parking lot for 2,000 cars and the PATH train station. The floors within the structure were reinforced concrete flat-slab construction supported by steel columns. The foundation of the tower consisted of massive reinforced concrete footings poured against the bedrock. Orthogonally placed steel beams were used to distribute the immense column loads, see Figure A.2.

A.3.2 Design and construction of the Twin Towers

A.3.2.1 General description

The Port Authority a public bi-state-agency of the two states New York and New Jersey, was the project developer and the owner of the World Trade Center. The architectural design was provided by Minoru Yamasaki and Associates and Emery Roth and sons. The structural engineers of the towers were Skilling, Helle, Christiansen, and Robertson.

The most known buildings of the World Trade Center complex were the two 110-storey twin towers. WTC1, the North Tower, was 1,369 feet (417 m) tall and WTC2, the South Tower, was 6 feet shorter. It had a roof height of 1,362 feet (415 m). On the top of the North Tower was a 360-foot (110 m) television antenna. The quadratic floor plan had a side length of 207 feet 2 inches (63.14 m) and chamfered corners of 6 feet 11 inches (2.11 m). Almost one acre (4,050 m²) was provided at each floor level. Within this quadratoe there was a rectangular service core which was 137 feet (41.76 m) long and 87 feet (26.52 m) wide. This service core contained 3 exit stairways, 99 elevators and 16 escalators, see Figure A.3.
Appendix A – The World Trade Center

A.3.2.2 Structural design

The structural design was almost identical for both towers. The service core of the North Tower WTC1 was oriented from west to east, whereas the South Tower WTC2 was oriented from north to south. The fact that the buildings influenced the wind loads on each other was considered in the structural design of the lateral force resisting system and lead to a somewhat different design.

The vertical fenestration was characteristic for the towers. It was proposed by the architect to give the people a sense of security. The engineers decided to use the closely spaced **perimeter columns** to resist the horizontal forces, see Figure A.3. At each side of the quadratic floor plan there were 59 perimeter columns and an additional column in the chamfered corners.

Each perimeter column consisted of four plates, which were welded together to form an approximately 14x13.5 inch (356 x 343 mm) rectangular hollow core section. The central spacing of the columns was 3 feet 4 inches (1.02 m) and at each floor the columns were interconnected by 52 inches (1.32 m) deep spandrel plates so that a Vierendeel girder was obtained. By assembling columns and spandrels, a web was obtained, which formed a perforated 207 feet 2 inches (63.14 m) square hollow section, see Figure A.4 and A.5. In the structural design this perforated hollow section was used to bear the significant wind loads to which the structure was subjected. Three 3-storey tall columns were combined with three spandrel plates to form a prefabricated module, see Figure A4 and A.5.

This modular prefabrication was a key factor for a rational and fast construction of the perimeter wall. The modules were arranged in a staggered way, and they were connected at the sides by high-strength bolted shear connections at mid-span. For the perimeter wall, twelve different grades of steel were used ranging from 42 ksi (290 MPa) to 100 ksi (690 MPa). The thickness of the column plates varied, as well. At the lower floors, three of the perimeter columns were merged together to form a single column, which permitted to have large openings for the buildings’ entrances.

**The floor construction outside the core** consisted of a composite structure with steel panels and concrete. These floor panels were 60 feet (18.3 m) or 35 feet (10.7 m) long and 20 feet (6.10 m) wide, see Figure A.6 to A.9. The floor panels consisted of a profiled metal deck, which were supported by pairs of trusses. The diagonals of the trusses were extended over the upper horizontal truss in order to obtain a shear stud behavior and therefore a composite steel deck after the concrete was poured onto the deck, see Figure A.6. The spacing between the trusses was 6 feet 8 inch (2.03 m). Due to the rectangular shape of the core, the span of the floor construction between the core and the perimeter column in WTC1 was 60 feet (18.3 m) in north-south direction and 35 foot (10.7 m) in west-east direction, see Figure A.9. At WTC2, the spans were arranged vice versa due to the pivoted core. Transverse trusses were spaced at 13 feet 4 inches (4.06 m) and intermediate deck support angles were spaced 6 feet 8 inch (2.03 m) between the transverse trusses, see Figure A.8 and A.9. This floor supporting system acted like a grillage that distributed the load to the various core and perimeter columns. At every second perimeter column the truss top chords were supported by seats as indicated in Detail A of Figure A.6. These seats were welded against the spandrel plates. At the core, the trusses were supported by seats connected to a girder, which in turn was supported by the core columns. In order to reduce the wind induced swaying, approximately 10’000 viscoelastic damping devices were installed as indicated in Detail A of Figure A.6. The devices were installed between the lower chord of the truss and the perimeter column. Flat bars provided with shear studs
The floor construction inside the core consisted of 5 inch (12.7 cm) concrete fill on metal deck supported by a floor framing system. These were supported by wide flange shape and box-section columns. Some of these columns had a cross-section of 14 inches (35.6 cm) by 36 inches (91.4 cm).
Appendix A – The World Trade Center

Figure A.6  Floor truss, from [58].

Figure A.7  Configuration of floors panels, from [80].
Between the 106th and the 110th floor an **outrigger truss system** was set in place. Diagonal bracing members together with the perimeter and core column formed an outrigger truss system. The truss system consisted of 10 truss lines; 6 in the longitudinal direction of the core and 4 perpendicular to these, see Figure A.10. The outrigger system had several functions. Firstly, it provided additional stiffness of the perimeter wall for wind resistance, and secondly it activated some of the core’s deadweight against horizontal loading by linking the core and perimeter walls. Thirdly, differential vertical deformations between the perimeter wall and the core were reduced by the outrigger truss system and finally, it provided the support of the transmission tower atop of WTC1.
The fire protection system consisted of several fire protection features such as compartmentation, fire detection system, notification system, suppression system, smoke management system and structural protection. The passive fire protection of WTC1 up to the 39th floor consisted of a spray applied product containing asbestos. Later, the material containing asbestos was either sealed or removed. The average thickness of the spray-applied fireproofing on steel trusses was \(\frac{3}{4}\) inch (1.9 cm). In the mid-1990s, it was decided to upgrade the fire protection by increasing the thickness to 1-1/2 inches (3.8 cm). This was done when floors were vacant. On September 11, 2001 the entire impact zone of WTC1 had been upgraded, however only one floor – the 78th floor – of the impacted area of WTC2 had an upgraded fire protection. Spandrels and girders were specified to have a 3-hour rating. Stair and elevator shaft were protected by a wall system consisting of gypsum boards, which had a 2-hour fire rating. The vertical compartmentation was provided by the floor slabs and the horizontal compartmentation varied throughout the building. In the beginning, the suppression system did not have an automatic sprinkler system. This was installed around 1990. The buildings had stand pipes throughout each of the three stairways. At each floor and in each stairway a 1.5 inch (3.8 cm) hose line and 2 extinguishers were available.

Emergency exits were provided by three stairways located in the central core of the buildings. Two of the stairways were 44 inches (1.12 m) wide and went up to the 110th floor and the third stairway had a width of 56 inches (1.42 m) and went up to the 108th floor. The stairways were not arranged in shafts running down from the top to the bottom. Therefore, people moving down the stairways had to pass through transfer corridors at different levels. After the bomb attack in 1993 the stairways were upgraded with battery-operated emergency lightning and photo luminescent paint to facilitate the emergency egress.

The elevator system was one of the innovations, which allowed the construction of the towers. The invented sky lobby system consisted of 99 elevators in each of the Twin Towers. 23 express lifts for 55 persons interconnected the concourse level with the sky lobbies at the 44th and 78th, see Figure A.11 and Figure A.12. From the concourse level and the sky lobbies 24 elevators served a third of the building so that 3 elevators were operating in a single shaft. Next to these elevators there were 4 cargo lifts. Next to a high velocity another advantage of the elevator system was that it allowed for 75% of the space to be rentable rather than 62%, which at that time was the best a skyscraper could achieve.
A.3.3 Innovations and special studies

Many people associated the silhouette of the WTC with the skyline of New York City. The Twin Towers also stood for innovation in the high-rise structural engineering and construction community. Among others, the following innovations and investigations were undertaken in connection with the design and construction of the WTC.

1) For the design of the towers the first comprehensive, boundary layer wind tunnel study were carried out.
2) For the first time a study of human sensitivity to building motion was carried out.
3) For the first time the perimeter wall was used as a perforated rectangular hollow section to resist the enormous wind loads.
4) The towers were the first buildings with mechanical damping units to reduce wind-induced swaying motion.
5) For the first time prefabricated multiple column and spandrel wall panels were used in buildings.
6) For the first time a divided lift system was established leading to sky lobbies, which are interconnected by 23 express lifts.

A.4 September 11, 2001

On Tuesday, September 11, 2001 the airplane of American Airline Flight 11 was hijacked by terrorists after take off from Boston’s Logan International Airport at 7:59 EST. Approaching from the north, the plane flew over Midtown Manhattan and hit the North Tower WTC1 at 8:46 EST. 92 people were on board. The second airplane United Airlines Flight 175 departed from Boston at
8:14 EST and approached Manhattan from the south. At 9:03 EST, this plane hit the South Tower WTC2. On board were 65 people. Both planes were Boeing 767-200ER loaded with fuel for a transcontinental flight to Los Angeles.

### Table A.1  Timeline of September 11, 2001, from [58].

<table>
<thead>
<tr>
<th>Start Time&lt;sup&gt;2&lt;/sup&gt;</th>
<th>Signal Duration</th>
<th>Magnitude (Richter Scale)</th>
<th>Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>8:46:26 EDT (12:46 UTC)</td>
<td>12 seconds</td>
<td>0.9</td>
<td>WTC 1 (the north tower) was hit by American Airlines Flight 11, a hijacked 767-200ER commercial jet airliner.</td>
</tr>
<tr>
<td>9:02:54 EDT (13:02 UTC)</td>
<td>6 seconds</td>
<td>0.7</td>
<td>WTC 2 (the south tower) was hit by United Airlines Flight 175, also a hijacked 767-200ER jet.</td>
</tr>
<tr>
<td>9:59:04 EDT (13:59 UTC)</td>
<td>10 seconds</td>
<td>2.1</td>
<td>WTC 2 began collapsing after 56 minutes, 10 seconds. Large debris from the collapse fell on WTC 3 and WTC 4, 130 Cedar Street, 90 West Street, and Bankers Trust. WTC 3 suffered a partial collapse. Fire was initiated in WTC 4 and 90 West Street.</td>
</tr>
<tr>
<td>10:28:31 EDT (14:28 UTC)</td>
<td>8 seconds</td>
<td>2.3</td>
<td>WTC 1 began collapsing after 102 minutes, 5 seconds. Large debris from the collapse fell on WTC 3, 5, 6, and 7; the Winter Garden; and the American Express (World Financial Center 2) building. WTC 3 collapsed to the 3rd floor, and fires were initiated in WTC 5, 6, and 7.</td>
</tr>
<tr>
<td>17:26:33 EDT (21:26 UTC)</td>
<td>18 seconds</td>
<td>0.6</td>
<td>WTC 7 began collapsing.</td>
</tr>
</tbody>
</table>

<sup>1</sup> Based on seismic recordings made by the Lamont-Doherty Earth Observatory of Columbia University, 34 kilometers north of the WTC site.

<sup>2</sup> EDT = Eastern Daylight Time; UTC = Coordinated Universal Time. Times cited in this report are based on these times, rounded to the nearest minute.

The Boeing 767-200ER has a wingspan of 156 feet 1 inch (47.6 m) and carries a maximum of 23'980 U.S. gallons (90'770 l) of fuel. Its maximum take off weight is 395'000 pounds (179 t) and it typically flies at a speed of 539 mph (867 km/h). In [58], the speed of impact into the North Tower is estimated to be 470 mph (756 km/h) and the impact speed into the South Tower is estimated to 590 mph (950 km/h). Kausel [90] assessed the airplane speed to 429 mph (691 km/h) and 503 mph (810 km/h), respectively. 56 minutes after the South Tower got hit; it collapsed at 9:59 EST. Then, 30 minutes later, at 10:29 EST, the North Tower collapsed, 1 hour and 43 minutes after it got hit. Debris from the towers fell onto surrounding buildings and damaged them. Furthermore, burning debris ignited fires in these buildings. The collapse of WTC2 led to an immediate collapse of the middle part of WTC3 the Marriott World Trade Center Hotel. Parts of the collapsing WTC1 fell on WTC3, as well, so that after the collapses only a small part of WTC3 still stood. WTC4, WTC5 and WTC6 burned out completely and suffered significant partial collapse. The 47-storey WTC7 building burned for 7 hours and collapsed at 17:20 EST, see Table A.1 and Figure A.13.
Figure A.13  Seismic records, from [58].

A.5  Structural response of the buildings

The present section summarizes the action to which the two towers were subjected on September 11, 2001 and the resulting response of the buildings. As already reported, both towers were constructed similarly and both were hit by a Boeing 767-200ER; however, the airplanes hit the towers at different floors, at different speeds and the core’s orientation was different, as well. Therefore, the two towers did not respond in the same way when they started to collapse. For this reason, a subsection is devoted to each tower’s structural response.

A.5.1  WTC1 – North Tower

A.5.1.1  Initial damage from airplane impact

On Tuesday September 11, 2001 at 8:46 EST the American Airline Flight 11 hit WTC1, the North Tower of the WTC complex, at the north side. The airplane entered the building between the 94th and 98th floor causing significant damage to the facade. The building was hit central, see Figure A.14. The damage to perimeter columns also led to failure of floor supports, which in turn led to partial collapses of floors in the affected area. The building performance study [58] estimates that, due to the initial impact of the airplane, 31 up to 36 columns from the north face were damaged. It is estimated that floors partially collapsed over a length of approximately 65 feet (19.8 m). Certainly, the central core was damaged, as well. However, the building performance study could not determine to what extend this happened. Interviews with eyewitnesses, which were at the 91st floor, directly under the impact area reported that there was extensive building debris in the eastern area of the central core. Therefore, the building performance study assumes that portions of the central core framing
immediately collapsed, as well. Eyewitnesses also reported seeing debris from partition walls in the western stairways. Therefore, the Building Response Study suggests a possible structural damage in the north western part of the core framing system. Furthermore, it is reported that debris from the intruding airplane traveled completely through the building. Life jackets and seats were found on the roof of the Bankers Trust/Deutsche Bank building and a part of the landing gear was found at the corner of West and Rector Street, 5 blocks south of the WTC complex. The fact that the tower did not collapse immediately, shows that it was able to carry the dead load although approximately more than 50% of its northern columns were damaged; this although the estimated airplane impact energy was higher than assumed in the structural design. The building was highly redundant and the columns immediately above the impact area were transformed into hangers. The loading was transferred via Vierendeel behavior to adjacent columns; it was also transferred via the outrigger truss system into the central core. A first structural analysis carried out by the Building Performance Study team concludes that the most heavily loaded columns were probably near to but not loaded in excess of their ultimate capacity. It was also found that the part above the impact area tilted towards the impact zone and subjected the remaining columns to additional stresses.

Due to the airplane impact, the building was seriously damaged; however, it remained standing and carried the dead load. The structure would probably not have resisted an additional extreme loading like heavy wind or an earthquake but probably could have been rehabilitated unless it would have been subjected to a developing fire.
A.5.1.2 Development of fire

By studying video recordings, it can be seen that the airplane first entered the building prior to the development of flames at the exterior of the building. Therefore, [58] suggests that the fuel of the airplane – in [58] it is indicated that both airplanes had about 10’000 gallons (38’000 l) fuel on board – was spread all over the damaged impact area and an explosive cloud developed. The ignition of this cloud led to a rise of pressure and large fireballs. It is also assumed that some of the fuel ran down into elevator shafts.

Video recordings of WTC2 show that three fire balls came out of the building, grew slowly and reached their full size after about 2 seconds. The fireballs had a diameter greater than 200 feet (61 m); therefore, they were larger than the building width. Such fireballs are formed if jet fuel is dispersed and flames travel through the jet fuel/air mixture. In [58], the amount of fuel consumed by these fire balls is estimated to 1’000 and 3’000 gallons (3’800 – 11’400 l). It is assumed that the same amount is consumed from the fireballs at WTC1. Due to the fact that the fireballs did not explode or generated shock waves, it is unlikely that the external fireballs created significant structural damage. If it is assumed that 3’000 gallons of jet fuel have been consumed by the fireballs, and if half of the remaining fuel ran into the shafts, then approximately 4’000 gallons jet fuel were available at the impact floor. According to [58], 10’000 gallons of jet fuel require 5 minutes for combustion, when enough air is provided. Therefore, it can be assumed that the total amount of jet fuel was consumed within the first minutes but it ignited fires all over the impact area.

In the same report, the energy output of the fire was estimated by using an advanced fluid dynamic fire model to generate the smoke plume of the fire. This energy output is estimated to 1-1.5 GW for each of the towers; for comparison, 1.17 GW is produced by Switzerland’s largest nuclear power plant. The ceiling gas temperature is estimated between 900 and 1100°C. In order to burn for 2 hours, such a fire requires a fire load which is be provided from 1 to 2 floors of the WTC. The air required for such combustion is in the order of 600’000-1’000’000 cu ft per minute (17’000 – 28’300 m³/min) and was provided by the damaged north face of the building and broken windows. It is assumed that the manual and automatic fire suppression system was damaged by the airplane impact and even if they would have worked they would have been ineffective.

A.5.1.3 Structural response to fire

The initial damage due to the airplane impact degraded the structure. Columns carried the load from other failed columns and the floor framing carried additional loads from floors above, which were partially collapsed. In addition, due to the impact, flying debris and the fireballs, the spray-applied fireproofing might have been damaged. With the development of fire and the raise of air temperature, the structure was additionally loaded and at the same time its strength was reduced. In [58], several possible failure modes were identified leading to the initiation of the progressive collapse. Here, three different mechanisms are listed and any combination of them might have led to the progressive collapse.

1) An increased temperature expand the floor framing and the slabs leading to additional column eccentricities.
2) Failed floor slabs increase the unsupported length of columns and reduced the resistance against buckling.
3) An increased steel temperature reduces the yield stress of the steel.

A.5.1.4 Collapse mechanism

8·10⁹ Joule of the total 4·10¹¹ Joule of potential energy was stored above the impact area. After the collapse started, this potential energy was transformed into kinetic energy. The enormous mass impacted onto the floor below, which immediately collapsed. The falling mass increased and accelerated, which led to a progressive collapse of the whole building. As the floors collapsed, they left free standing perimeter wall segments and possibly central core columns. With increasing unsupported length of the columns they buckled at their bolted connections and fell down.

In [58], it is indicated that the transition tower atop of WTC1 at first moved down and slightly laterally before the exterior wall started to move. This suggests that the failure began in the central core area, which is consistent with the reported debris from the 91st floor. Due to the fact that core
columns were not designed for wind loads, they were closer to their ultimate capacity than the exterior columns. Furthermore, additional load was transferred to the central core via the outrigger truss system after the plane impact. Once the movement began, the entire mass above moved downwards leading to a successive collapses of floors.

**A.5.2 WTC2 – South Tower**

**A.5.2.1 Initial damage from airplane impact**

On September 11, 2001 at 9:03 EST, the United Airlines Flight 175 hit the South Tower WTC2. The airplane entered the building between the 78th and 84th floor of the south facade. The impact zone was near the southeast corner and 27 to 32 columns were damaged over a range of 5 stories at the south face of the building. Partial collapses of floors, which were supported from seats fixed to the perimeter wall, occurred over a length of around 70 feet (21.3 m). It is likely that columns of the southeast core were damaged because they stood in the travel path of the fuselage and port engine. Some of the airplane debris traveled through the building. The landing gear crashed into the roof of a building six blocks away from the WTC complex and a jet engine was found at the corner of Murray and Church Street. Next to the south façade, the debris damaged the southeast corner of the central core, the floor construction and the north, east and west walls.

Some differences between the airplane impacts into the two towers should be emphasized. Firstly, the plane hitting WTC2 traveled at a much higher speed of 590 mph (950 km/h) rather than 470 mph (756 km/h). Hence, the kinetic energy related to the impact into WTC2 was much higher. Secondly, the area of impact at WTC1 was in the middle of the facade whereas WTC2’s impact area was close to the southeast corner. Thirdly, the core of WTC2 was oriented in a way that the airplane had to travel 35 foot (10.7 m) across the floor until it hit the core. At WTC1, the distance was 60 feet (18.3 m). Moreover, the impact zone of WTC2 was much lower than the one of WTC1 leading to larger loads. Finally, the impact area of WTC1 had a spray applied passive fire proofing with a thickness of 1-1/2 inches (3.8 cm). At WTC2 only the 78th floor had such a fireproofing. The remaining floors of WTC2’s impact area had a ¾ inch (1.9 cm) fire proofing. These differences might have led to the fact that WTC2 collapsed earlier than WTC2.

Figure A.16 Damaged south and east facade of WTC2, from [58].
A.5.2.2 Development of fire

The development of the fire, which followed the impact, was similar to the development of the fire in WTC1. The most intensive fires occurred along the north face of the building.

A.5.2.3 Initiation of collapse

The same failure mechanisms as discussed for WTC1 may have caused the collapse of WTC2. [58] suggests that the collapse began with a partial collapse of the floor in the southeast corner around the 80th floor. Following this, the whole floor collapsed blowing out a line of dust at the sides. After that the columns of the east side started to buckle. They started at the southeast side and progressed to the north. Then the mass above this floor started rotating towards east and south. This was the trigger event for the progressive collapse of the whole building. In [58], it is noted that failure of columns at the southeast corner of the central core might have been the initiating event of this progressive collapse. It appears that as the upper part of the building fell to the south and east, the lower part fell to the north and west. Therefore, WTC3 the Marriott Hotel was hit by WTC2.

A.5.3 Summary

The Twin Towers sustained enormous damages due to the airplanes’ impacts. Although degraded, these towers were able to remain standing. However, they possible would not have survived another extreme load event like an extreme wind or an earthquake, see [58]. The proceeding fires, which were ignited by the impacting airplane, were such extreme load events. Despite these extreme conditions, WTC2 remained standing for 56 minutes and WTC1 for 1 hour and 43 minutes. This permitted the people below the impact area to evacuate the building. Among others studies, the structural performance of the WTC and its subsequent collapse was also analyzed by [10], [160], [89] and [116].
B Detailed consequence assessment of the WTC failure

A summary of the consequences following the failure of the World Trade Center towers was already presented in Section 2.4. The aim of the present annex is to summarize the consequence assessment in more detail. First of all, rescue and clean-up costs are identified. Then the property losses are assessed followed by an assessment of the impact to the economy, the consequences due to fatalities, environmental consequences and the loss of cultural assets. The annex closes with remarks to changes in risk perception and the total economic consequences to the society.

B.1 Rescue and clean-up costs

Immediately after the first plane hit Tower One of the World Trade Center, the first organizations were alarmed and started the evacuation operation, see e.g. [110]. According to [100], this initiated the largest rescue operation in the history of the United States of America. Later other federal, state, city and non profit organizations and agencies joined the rescue efforts. Among others, there were the American Red Cross, New York Fire Department (FDNY), the New York Police Department (NYPD), the Federal Emergency Management Agency (FEMA), the Urban Search and Rescue teams (US&R) and the Port Authority Police Department (PAPD).

The alarm, which was activated after the first plane hit WTC1, marked the start of the evacuation and the rescue effort. While the evacuation was terminated by the collapse of the building, the rescue effort continued. 13 days later, on September 24, 2001, Major Giuliani declared the official end of the rescue period [132]. The twilight ceremony, at which a 58-ton steel column was removed on May 28, 2002, marks the end of the clean-up effort [50].

In [44], it is reported that during the clean-up, more than 1.7 million tons of steel and other debris have been removed, whereas [157] reports that 1.8 million tons of debris were carried away by 100'000 trucks. It further states that 3.1 million man hours have been spent during the clean-up. Until December 31, 2001, the workers worked round-the-clock. Thereafter, a 10 hours six day workweek was set up [44].

In [198], it is reported that during the first week after the failure, there were an estimated number of 10'000 workers at the WTC site – “ground zero”. This number dropped to less than 2'500. In addition to fire and policemen, there where 1’500 iron workers and 400 machine operators at the site, as well as carpenters and other laborers. The appointed construction enterprises were: Bovis Lend Lease, Bechtel, AMEC and Tully Construction Co. Inc. and Turner Construction Co. [45].

[198] reports that the rescue and clean-up work was in fact a dangerous undertaking. Smell of decay was raising concerns about infectious disease control, glass from the broken windows of surrounding buildings threatened to fall down onto passing workers; damaged scaffolding of neighboring buildings represented serious hazards. In addition, the debris pile at ground zero was hot with temperatures of more than 400°F (200°C). Also the structural integrity of the debris pile, as well as the remaining WTC buildings and the bathtub – the substructure of the WTC – represented a high hazard potential [198]. After the collapse of a basement, the work was partially halted, see [135]. Next to these hazard potentials, there were hazardous materials stored in the World Trade Center; among them were 200’000 pounds (90.7 t) of freon used as refrigerant for heating and ventilation systems, an unknown quantity of blood products, 3’600 pounds (1.63 t) of lead acid batteries and a significant amount of ammunition [198]. [76] reports that lead was present at the World Trade Center.
inside computers corresponding to 200’000 – 400’000 pounds (90.7 – 181 t). Furthermore, 10 to 25 pounds (4.5 – 11.3 kg) of mercury was present in 500’000 fluorescent lamps. In addition, more than 130’000 gallons (492 m³) of oil and insulation fluid were stored in the basement of WTC7, which housed the Con Edison substations. Moreover, dioxins, furans, benzene, PCB and asbestos could be detected [76].

It seems unnecessary to differentiate the costs further into evacuation, rescue and clean-up costs, because the costs related to evacuation were negligibly low. Furthermore, it is rather difficult to separate rescue and clean-up costs as in both periods, almost the same work has been done, with the difference that during the clean-up phase attention is paid to possible survivors.

Several sources have published estimates of the clean-up costs. As early as November 8, 2001, the clean-up costs were estimated to 2.5 billion USD [44]. In March 2002, the costs were estimated to be below 1.2 billion and in May the costs were reported to be 750 million USD [113]. Also a number of 650 million was reported [157]. Finally, two years after the attack, a New York Times article [140] indicates that 582 million USD were spent for clean-up.

<table>
<thead>
<tr>
<th>Table B.1 Rescue and clean-up costs according to [59].</th>
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<tbody>
<tr>
<td><strong>Rescue &amp; Clean up costs</strong></td>
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<tr>
<td><strong>FEMA</strong></td>
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<tr>
<td>US&amp;R &amp; Mission Assignments</td>
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<tr>
<td>297'711'792</td>
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<tr>
<td>Debris removal</td>
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<td>Debris removal</td>
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<tr>
<td>437'000'000</td>
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<tr>
<td>Landfill</td>
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<td>90'000'000</td>
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<tr>
<td>Insurance</td>
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<td>96'000'000</td>
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<td>Debris removal</td>
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<tr>
<td>623'000'000</td>
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<td>Ferry</td>
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<td>33'000'000</td>
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<td>Train</td>
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<td>15'000'000</td>
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<tr>
<td>Road repair</td>
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<td>9'100'000</td>
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<tr>
<td>Pedestrian bridge</td>
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<td>5'200'000</td>
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<td>Bus</td>
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<td>Traffic operation</td>
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<td>315'000</td>
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<td>Debris removal</td>
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<td>636'150'000</td>
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<td>Debris removal</td>
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<tr>
<td>Indoor residential cleaning</td>
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<td>80'000'000</td>
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<td>Exterior buildings</td>
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<td>NY University air monitoring &amp; cleaning</td>
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<td>5'900'000</td>
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<td>Air testing in schools</td>
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<td>2'900'000</td>
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<td>Cleaning in schools</td>
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<td>4'100'000</td>
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<td>Pace University air monitoring</td>
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<td>103'300'000</td>
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<td>Debris removal</td>
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<tr>
<td>New York Fire Department</td>
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<tr>
<td>Overtime</td>
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<td>105'600'000</td>
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<td>Death benefits</td>
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<td>103'900'000</td>
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<tr>
<td>Destroyed vehicles</td>
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<td>28'300'000</td>
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<td>Cleaning fire apparatus</td>
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<td>2'300'000</td>
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<tr>
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<tr>
<td>241'600'000</td>
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<td>New York Fire Department</td>
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<td>Overtime</td>
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<td>295'400'000</td>
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<td>Destroyed vehicles</td>
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<td>5'000'000</td>
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<td>300'400'000</td>
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<td>New York Police Department</td>
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<td>28'800'000</td>
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<tr>
<td>Destroyed vehicles</td>
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<td>11'800'000</td>
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<td>Debris removal</td>
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<td>44'600'000</td>
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<td>Debris removal</td>
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<tr>
<td>Total</td>
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<td>1'674'226'792</td>
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A detailed list of clean-up costs and rescue costs is given in [59]. According to this FEMA report, the clean-up costs excluding insurance for the clean-up effort is 527 million USD. If costs for insurance of the debris removal are included, then the debris removal amounts to 623 million USD. Expenses for cleaning and air monitoring are 103 million USD, which results in clean-up costs of 726 million USD. Costs for the National Urban Search and Rescue Response System amount to 298 million USD, expenses associated to the Fire Department of the City of New York total to 241 million USD and the Port Authority Police Department obtained 44.6 million USD from FEMA. The New York City Police Department received 300 million USD from FEMA. Proportions of this have to be addressed as special expenses for counterterrorism because after the terrorist attack the police department showed an increased presence in the public to safeguard the population. Finally, the rescue and clean-up costs amount to 1.67 billion USD, of which the clean-up costs represent 43.4%.
B.1.1 Summary
The evacuation, rescue and clean-up costs are determined by several factors. For instance the number of people to be rescued and evacuated plays a role, as well as the specific hazard scenario. Furthermore, the amount of debris to be removed and the area which is polluted and contaminated influences the clean-up costs. For ordinary rescue and clean-up efforts it is possible to calculate the total rescue and clean-up cost by using unit costs and the corresponding quantity. For the case of the WTC failure however, it was difficult to predict the total rescue and clean-up costs, because much uncertainty was associated to it. The estimates varied strongly with time as information had become available. For instance, the structural integrity of the bathtub was such an uncertain factor. If the bathtub had collapsed the water of the neighboring Hudson River would have flooded the substructure. This would have complicated the clean-up and additional expenses would have followed. Therefore, a simple model seems appropriate, which simply models the rescue and clean-up costs for a given type of structure by referring the clean-up and rescue costs to the reconstruction costs. One may also relate these costs to the gross floor area of the considered building. Considering the WTC Twin Towers’ gross floor area and their reconstruction costs, which are determined in B.2.1, rescue and clean-up costs amount to 36% of the reconstruction costs or 175 USD/sq ft.

The variation of rescue and clean-up costs with time seems to be more appropriately described by the Construction Cost Index (CCI) than by the Building Cost Index (BCI) because rescue efforts and clean-up are labor force intensive.

B.2 Property losses
Failure of high-rise buildings may lead to damages or even complete collapses of neighboring buildings or infrastructural facilities. Property losses also have to account the loss of the inventory which is contained within these buildings and facilities. The failure of the WTC Twin Towers represents such a case, where surrounding buildings and infrastructural facilities and their inventory have been damaged or even destroyed. This is analyzed in the following.

B.2.1 Buildings
Losses due to failed or damaged buildings and infrastructural facilities can be assessed in terms of repair or reconstruction costs. Generally, if a building or an infrastructural facility fails or is damaged, a decision has to be made whether to repair or reconstruct it. The alternative options are to demolish the remaining structure and let the land lie idle or to reuse it for the same or a different purpose. Within the consequence assessment procedure for high-rise buildings, it can be assumed a priori that buildings and facilities will be reconstructed or repaired because they contribute essentially to the wealth of individuals and/or the society. Then, repair and/or reconstruction costs are implied. In the case when a building or a facility will not be reconstructed, then its owner has to be compensated by the actual cash value [183], which has to be estimated by an independent real estate appraiser [84]. The actual cash value might differ from the reconstruction costs; nevertheless, it should be of the same magnitude.

In case of the failure of the WTC Twin Towers, the lease contract for the towers establishes both a concomitant right and an obligation for the lessees to reconstruct the leased buildings after a failure, at their own costs and expenses, whether or not the damage is covered by insurance payment, see [154]. Hence, the leaseholder is obligated to reconstruct.

B.2.1.1 Construction costs
In 1966, the Port Authority announced that the cost estimates for the construction of the WTC increased to 575 million USD. The initial construction costs of 350 million were very optimistic. This number was obtained by assuming ten million square feet of space to construct at a price of 35 USD per square foot. At that time this unit price was the price for an ordinary structure of 40 stories in midtown Manhattan [74]. Finally, the construction of the whole World Trade Center cost 1.5 billion USD [178]. In [211] it is reported that until 1992 the Port Authority invested 1.29 billion USD in the World Trade Center. In the beginning of 2001 the WTC complex was noted in the Port Authority’s budget with 1.13 USD [154]. In [169], the construction costs for the WTC Twin Towers is given as 900 million USD, which results in a unit price of 94.5 USD/sq ft.
Appendix B – Detailed consequence assessment of the WTC failure

B.2.1.2 Reconstruction costs

In order to evaluate the reconstruction price 31 years later, the Building Cost Index (BCI) may be utilized. In 1968 the BCI was at 721 points and in 2001 it was at 3’574. This implies that during that period construction prices increased by a factor of 4.96 corresponding to an equivalent annual price rise of 4.97%. Following this approach, the reconstruction of the WTC towers would costs around 470 USD/sq ft. The BCI is analyzed more thoroughly in Annex C.3.

For the construction of a high-rise building in year 2002, Swiss Reinsurance published a unit price of less than 300 USD/sq ft [183] and in [139] a unit price of 350 USD/sq ft is indicated. An American construction enterprise reports that construction costs for high-rise buildings with standard interior range from 350 to 400 USD per square foot gross floor area. Rehabilitation costs for a standard interior range from 110 to 130 USD/sq ft and rehabilitation of the facade costs from 35 to 55 USD/sq ft. Rehabilitation costs for severely damaged facades may amount to the reinstatement value, which ranges from 85 to 95 USD/sq ft. A unit cost of 600 USD/sq ft is indicated for construction costs in [118]. However, this number also considers 180 USD/sq ft for the cost of the land so that the construction costs for the structure only amount to 420 USD/sq ft. Unit costs may also be derived from information published about the new WTC7 building.

B.2.1.3 WTC7

In addition to WTC1 and WTC2, also WTC7 collapsed completely on September 11, 2001. The 47 floor office building was constructed in 1987 and was 570 ft (174 m) tall [176]. The construction of the replacing building commenced in 2002. It will have a parallelogram shaped floor plan rather than a trapezoidal one, which had the old WTC7, see Figure B.1. This allows Greenwich Street to continue up to the WTC complex, which previously ended in front of WTC7 [46]. Due to this, the floor plan of the new WTC7 is much smaller than the former WTC7. In order to compensate for the lost space in the floor plan, the new WTC7 will be taller. It will be 750 ft (229 m) tall with 52 floors resulting in 1.7 million sq ft, which is still less than 2 million sq ft, the size of the old WTC7. 42 floors are dedicated as office floors, whereas the lower 10 floors will accommodate the new substation of Con Edison [175]. The construction will be finished in late 2005 or early 2006. Up to then, 700 million USD will be invested in this building [136]. From these numbers expected unit costs of 412 USD/sq ft are derived.

B.2.1.4 Damaged buildings

Based on inspections from the Structural Engineers Association of New York [43], the Building Performance Study Team published the extent of damages to buildings in its report [58]. Mapping of damaged buildings can also be found in [186] and [99]. In Table B.2 the buildings are differentiated into destroyed, severely damaged and moderately damaged buildings. Based on data from [186] and [99] it is found that 14 million sq ft gross floor area have been destroyed, 8.5 million sq ft suffered severe damage and around 20 million sq ft was moderately damaged. Figure B.1 indicates the spatial distribution of the damage. The damage categories can also be represented by damage factors. The assessment of an appropriate damage factor is simple for the extreme cases of undamaged and completely destroyed buildings. For the case of a completely destroyed building, this factor is one and the damage factor for an undamaged building is zero. However, it may be difficult for the cases between those extremes. The Deutsche Bank building constitutes such a case.

B.2.1.5 Deutsche Bank building

The Deutsche Bank building is located at 130 Liberty Street just south of WTC2 and WTC4, see Figure B.1. Before Bankers Trust was acquired by the Deutsche Bank in 1999, the building was known as the One Bankers Trust Plaza. The building had 40 floors and is 565 ft (172 m) tall. Its construction ended in 1974 [177] and the building contains 1’415’086 sq ft of office space [186]. Deutsche Bank, owner of the heavily damaged building, filed a lawsuit against two of its insurers. From these insurers, Deutsche Bank claimed 1.05 billion USD from a total insurance claim of 1.7 billion USD to cover demolition, clean-up and business interruption costs. However, the accused insurers insisted that the building can be repaired and cleaned for 500 million USD. This is the half of the Deutsche Bank’s claim which is directly related to the building.

Regarding the structural integrity, the building can be repaired; but, it suffered additional mold infection and contamination from fires, see [209], [98] and [78]. The court would have had to decide
whether the damage factor is 50% or 100%, respectively. However, in February 2004 the bank
settled the dispute with its insurers [148] and will sell the building to the Lower Manhattan
Development Corporation, which will pay for the demolition together with the insurers. The property
will be integrated in the WTC redevelopment plans and it will become a new park together with an
underground garage and possibly a new office building.

B.2.1.6 Summary of building losses

Due to the attacks, 14 million sq ft gross floor area have been destroyed, 8.5 million sq ft suffered a
severe damage and around 20 million sq ft was moderately damaged. Considering a unit price of 500
USD/sq ft, the Comptroller of New York City estimates the loss of the WTC Twin Towers to
4.7 billion USD. Reconstruction of the other destroyed buildings will cost 2.0 billion USD.
4.3 billion USD is expected to be spent for the restoration of the majorly and moderately damaged
buildings. Finally, the costs to replace and restore the destroyed and damaged buildings are estimated
to total 11 billion USD [118].

Figure B.1  Spatial distribution of building damage, from [58].
Appendix B – Detailed consequence assessment of the WTC failure

Table B.2 Destroyed and damaged buildings.

<table>
<thead>
<tr>
<th>Destroyed buildings</th>
<th>Moderately damaged buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 WTC</td>
<td>14 Barclay St.</td>
</tr>
<tr>
<td>2 WTC</td>
<td>101 Barclay St.</td>
</tr>
<tr>
<td>3 WTC</td>
<td>125 Barclay St.</td>
</tr>
<tr>
<td>4 WTC</td>
<td>111 Broadway</td>
</tr>
<tr>
<td>5 WTC</td>
<td>174 Broadway</td>
</tr>
<tr>
<td>6 WTC</td>
<td>187 Broadway</td>
</tr>
<tr>
<td>7 WTC</td>
<td>189 Broadway</td>
</tr>
<tr>
<td>155 Cedar Street</td>
<td>1 Carlisle</td>
</tr>
<tr>
<td>5 Carlisle</td>
<td>47 Church St.</td>
</tr>
<tr>
<td>120 Cedar</td>
<td>90 Church St.</td>
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<tr>
<td>130 Cedar</td>
<td>10 Church St.</td>
</tr>
<tr>
<td>114 Liberty St.</td>
<td>110 Church St.</td>
</tr>
<tr>
<td>130 Liberty St.</td>
<td>120 Church St.</td>
</tr>
<tr>
<td>45 Park Place</td>
<td>10 Cortlandt</td>
</tr>
<tr>
<td>30 West Broadway</td>
<td>22 Cortlandt</td>
</tr>
<tr>
<td>90 West St.</td>
<td>110 Greenwich St.</td>
</tr>
<tr>
<td>140 West St.</td>
<td>2 WFC</td>
</tr>
<tr>
<td>2 WFC</td>
<td>3 WFC</td>
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<tr>
<td>125 Greenwich St.</td>
<td>224 Greenwich St.</td>
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<tr>
<td>1 Liberty Plaza</td>
<td>106 Liberty St.</td>
</tr>
<tr>
<td>110 Liberty St.</td>
<td>120 Liberty St.</td>
</tr>
<tr>
<td>124 Liberty St.</td>
<td>125 Cedar Street</td>
</tr>
<tr>
<td>9 Maiden Ln.</td>
<td>5 Carlisle</td>
</tr>
<tr>
<td>7 Park Place</td>
<td>47 Church St.</td>
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<tr>
<td>90 Trinity Pl.</td>
<td>90 Church St.</td>
</tr>
<tr>
<td>110 Trinity Pl.</td>
<td>10 Church St.</td>
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<tr>
<td>26 Vessey St.</td>
<td>110 Church St.</td>
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<tr>
<td>28 Vessey St.</td>
<td>120 Church St.</td>
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<tr>
<td>2 Wall Street</td>
<td>10 Cortlandt</td>
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<tr>
<td>60 Warren St.</td>
<td>22 Cortlandt</td>
</tr>
<tr>
<td>1 WFC</td>
<td>110 Greenwich St.</td>
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</tbody>
</table>

B.2.2 Infrastructural facilities

In addition to buildings, the surrounding infrastructure at the WTC site suffered extensive damages, as well. Gas and steam mains, as well as telephone and power lines were interrupted, roads were damaged and pedestrian bridges were destroyed. Moreover, the nearby Metro lines from the Metropolitan Transportation Authority (MTA) were severely damaged, as well as the Port Authority Trans-Hudson (PATH) railway. In the following, the damage to the infrastructure facilities are described and estimated monetarily.

B.2.2.1 Road and pedestrian bridges

The West Street, which borders the WTC site, suffered damage from the collapsing buildings. A 1’000 ft (305 m) long temporary section of West Street did cost 5 million USD. It is estimated that this road will be used approximately three to five years until a more comprehensive redevelopment plan is enacted. Reconstruction of a 250 ft long pedestrian bridge crossing West Street cost 3.3 million USD [47].

B.2.2.2 Metropolitan Transportation Authority subway

The damage to the Metropolitan Transportation Authority’s (MTA) system was severe but localized. The subway tunnel of line 1 and 9 collapsed between Barclay Street and Liberty Street. Furthermore, the subway station at Cortlandt Street was destroyed and the Rector Street station was damaged. Although there was no interruption in the MetroCard fare collection system, fare collection was suspended on September 11, 2001. At 10:20 EST that day all services were halted. At 12:48 EST the services were restored and by the day’s end 65% of the service was reestablished.

Initially, the repair costs of the damaged property was estimated to 1.7 billion USD but was later reduced to 855 million USD because the N and R lines below Broadway were not as severely damaged as initially expected. For the reconstruction of the tunnel, a 92 million contract was awarded to Tully Construction and Pegno Construction in May 2002 [103]. The contract includes a 1’200 ft (365 m) complete tunnel replacement including tunnel lightning, track and signal work [120].

B.2.2.3 Port Authority of New York and New Jersey

The restoration of the Port Authority Trans Hudson (PATH) system has initially been estimated to 850 million USD in 2001 [131]. Finally, the reconstruction of the PATH is planned within a 544 million USD program. From this amount 160 million USD are devoted to the restoration of the Exchange Place PATH station in New Jersey and the tunnels crossing the Hudson River. 300 million USD are used to construct a temporary PATH station at the WTC site. Before September 11, 2001, this station was the busiest of the whole PATH system with 65’000 passengers daily, see [153], [156] and [155]. Late in November 2003, the temporary PATH station at the WTC site was
finally reopened. In addition, the Port Authority lost one power substation worth 40 million USD and
the headquarter estimated to 150 million USD [131].

B.2.2.4 Electrical energy, gas and steam

The failure of the WTC towers also affected the electrical power, gas and steam transmission and
distribution system in Lower Manhattan. These systems are owned by Con Edison Incorporated. Con
Edison reports that two large areas of Lower Manhattan were without power supply from September
11 onwards. The first area is bounded by Dover Street in the north, William Street in the west, Wall
Street in the south and by the East River in the east. The second area is bordered by Thomas Street in
the north, Hudson River in the west and Broadway in the east down to the southern tip of Manhattan
[26]. Within six days, Con Edison restored full electrical power supply for the reopening of the New
York stock exchange. In less than eight days on September 19 at 3:51 EST, 1’900 Con Edison
workers restored the electrical power service to 98% of more than 13’300 affected customers by
laying more than 36 miles (58 km) of temporary electrical cables. Additionally, the gas service was
restored to more than 5’500 customers and steam service to 300 customers [27], [29] and [49]. The
costs associated to emergency response, temporary restoration and permanent repair of electric, gas
and steam transmission and distribution facilities are estimated to 430 million USD [30].

B.2.2.5 Telecommunication

The World Trade Center failure caused the interruption and damage of 300’000 phone lines and 5
switching stations. After the attack, the communication enterprise Verizon deployed 3’000
employees to Lower Manhattan to restore phone service to residents and businesses. This permitted
the New York Stock Exchange at Wall Street to reopen on September 17, 2001. Until September 24,
2001, Verizon restored services to two third of its affected customers or provided alternative
solutions, such as rerouting numbers to new locations [194]. 4’000’000 voice and data circuits were
constructed, reconstructed or rerouted. Verizon also erected 21 temporary cellular towers, provided
free wireless phones and free wireless pay phones on trailers and installed 18 new SONET
(synchronous optical network) fiber-optic rings.

Initially, the costs of Verizon and AT&T were estimated to 2.3 billion USD [118]. In 2001 until
December 31, 2001, Verizon recorded equipment losses, costs associated with service disruption and
restoration of 685 million USD. Moreover, Verizon experienced operating expenses of 112 and 130
million USD in 2002 and 2003, respectively, see [195], [196] and [197]. When no additional
expenses for the year 2004 are considered, then Verizon’s loss amounts to 927 million USD, which
should all be covered by its insurance.

Following the Twin Towers’ failure, Verizon transmitted 230 million calls through its network in the
New York City each day for at least a week following September 11, 2001. This is twice as much as
the normal daily volume of about 115 million calls. At the same time, AT&T handled 431 million
calls, which is 100 million more than ever observed before [7]. AT&T was affected from the impact
as well; however, not in the same magnitude and the corresponding losses have not been accounted
for separately.

B.2.2.6 Redevelopment of Lower Manhattan

According to the Lower Manhattan redevelopment plan, the temporary PATH station will be
replaced by a permanent PATH station situated in the WTC Transit Hub. This Hub will cost 1.7 to
2.0 billion USD. A 2’500 ft (762 m) long and 50 ft (15.2 m) wide underground concourse will
connect the WTC Transit Hub with the World Financial Center in the west and the Fulton Street
Center in the east. The Fulton Street Center will reconfigure and improve the existing Subway
stations and connect the subways more efficiently. This center will cost 750 million USD.
Additionally, it is envisaged to spend 1.7 to 2.0 billion on several development projects such as a 500
million USD bus facility or the restoration of West Street, Fulton Street and Greenwich Street.
Moreover, a 400 million USD rehabilitation of the South Ferry Terminal will help to improve
transportation in Lower Manhattan, see [126] and [137]. These infrastructural facilities were not in
place before September 11, 2001 and have to be seen as improvements to Lower Manhattan, which
are constructed in the same line as the reconstruction is done. Therefore, these costs are not
considered as consequences due to the WTC failure.
B.2.2.7 Summary infrastructure losses

By the failure of the World Trade Center towers, the infrastructure in its vicinity was severely damaged. Roads were damaged, pedestrian bridges destroyed, gas and steam mains were interrupted, as well as phone and power lines. In addition, the PATH station underneath the WTC complex was completely destroyed and its tunnels were flooded with water from the Hudson River. Furthermore, the Subway needed a 1`200 ft (365 m) long replacement tunnel for its lines 1 and 9. The complete loss of infrastructure is estimated to 2.95 billion USD, which is approximately 63% of the reconstruction costs for the Twin Towers. Table B.3 summarizes the infrastructural losses. 31% are attributed to the damaged telecommunication system. Compared to this, the costs due to the damage of West Street and the collapse of a pedestrian bridge seem to be negligibly small.

Table B.3 Destroyed and damaged infrastructure.

<table>
<thead>
<tr>
<th>Destroyed and damaged infrastructure</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Port Authority</td>
<td>Port Authority</td>
</tr>
<tr>
<td>PATH</td>
<td>544</td>
</tr>
<tr>
<td>Power substation</td>
<td>40</td>
</tr>
<tr>
<td>Headquarter</td>
<td>150</td>
</tr>
<tr>
<td>Verizon</td>
<td></td>
</tr>
<tr>
<td>Telecommunication system</td>
<td>927</td>
</tr>
<tr>
<td>Con Edison</td>
<td></td>
</tr>
<tr>
<td>Electric power, gas &amp; steam</td>
<td>430</td>
</tr>
<tr>
<td>MTA</td>
<td></td>
</tr>
<tr>
<td>Subway</td>
<td>855</td>
</tr>
<tr>
<td>Road &amp; Bridges</td>
<td></td>
</tr>
<tr>
<td>West Street reconstruction</td>
<td>5</td>
</tr>
<tr>
<td>Pedestrian bridge</td>
<td>3</td>
</tr>
<tr>
<td>Total</td>
<td>2'954</td>
</tr>
</tbody>
</table>

(in million USD)

B.2.3 Inventory

The Comptroller of New York City estimated the loss of inventory to 125 USD/sq ft, which is seen as an industry standard [118]. On this basis, the lost inventory is estimated to 3.2 billion USD [188]. In addition, 2.0 billion USD are assigned to the loss of the retail space’s inventory [118]. Considering both, the value of the lost inventory is estimated to 5.2 billion USD or 1.11 times the reconstruction costs of the two towers.

B.2.3.1 Gold and other inventories

In the basement of WTC4 there was a vault, which was used by the Bank of Nova Scotia. In that vault the bank stored 379’036 ounces (10.7 t) of gold and 29’942’619 ounces (849 t) of silver. The value was estimated by the Bank of Nova Scotia to 200 million USD. In November 2001, a team of 30 firemen and police officers secured and moved the assets so that the bank did not experience a loss [134].

In WTC6, the building named after the United States Customs, hosted evidence vaults, in which smuggled goods, drugs and weapons were stored. Furthermore, beneath this building, there was a fleet of government vehicles. Dozens of them were owned by the Secret Service. In total, the World Trade Center’s parking garage, which was situated in the basement, provided space for 2000 vehicles. No source was found which reported the value of these goods and the damage they sustained.

Table B.4 Property losses.

<table>
<thead>
<tr>
<th>Property Losses</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings</td>
<td></td>
</tr>
<tr>
<td>WTC Towers</td>
<td>4.7</td>
</tr>
<tr>
<td>Destroyed Buildings</td>
<td>2.0</td>
</tr>
<tr>
<td>Damaged Buildings</td>
<td>4.3</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>3.0</td>
</tr>
<tr>
<td>Inventory</td>
<td>5.2</td>
</tr>
<tr>
<td>Total</td>
<td>19.2</td>
</tr>
</tbody>
</table>

(in billion USD)
B.2.4 Summary property losses

As a result of the towers’ failures, property worth 19.2 billion USD was lost. This is more than four times the value of the Twin Towers, which is estimated to 4.7 billion USD. In addition, buildings worth 2.0 billion were destroyed and 4.3 billion USD are assigned to damaged buildings. The losses to the infrastructure are estimated to 3.0 billion USD and the lost inventory is estimated to 5.2 billion USD.

B.3 Influence on the economy

In addition to rescue and clean-up costs, property losses and consequences due to fatalities, the impact on the economy constitutes a major part of the overall socio-economic consequences.

In the case, when an adverse event affects a small number of businesses, the reduced turnover and benefit may be assessed together with additional costs and expenses. In order to model and assess these consequences, basis can be taken in micro-economics. However, when the number of affected businesses is large, this approach becomes impractical. In such cases, business losses can be modeled and assessed by concepts known from macro-economics. An introduction to macro-economics and its aggregates, such as the gross domestic product (GDP) is given in Annex C. There it is also discussed, why the GDP may be utilized for the assessment of economic consequences. The present section first gives an overview of the economy of New York City and analyzes the New York City demographically; then the business interruption costs are evaluated, which is also done for selected infrastructural facilities. Finally, the lost rents will be assessed.

B.3.1 Demography and economy of New York City

B.3.1.1 The United States of America

In 2001, the population of the United States of America was 288 million and produced a GDP of 10'065 billion USD. Hence, 4.7% of the global population produced 24.4% of the world’s GDP in purchasing power parity (PPP). Dividing the USA’s GDP by the number of persons living in the USA, a GDP per capita of 34'320 PPP USD is obtained. At the same time, the world’s GDP per capita was 7'376 PPP USD or 4.7 times less than the US GDP per capita, see [189].

Figure B.2 shows the frequency distribution of the USA population’s age. This is shown for each gender and the total population. After being roughly constant, the frequency distributions for both gender and the total population have a mode at 40 and then decline. The mean value of the male population is 34.5 year, for the female population 37.1 years. For the total population the mean age is 35.8. The standard deviation is 22.4 years. The higher mean value of the female population’s age is also a result of the higher life expectancy. This is also seen in the higher fraction of the female to the total population for ages larger than 35 years. For ages equal to or below 35 years, the male population preponderates. At its maximum, it is 7% higher.

The United States nominal GDP of 2002 amounts to 10'480.8 billion USD. Table B.5 summarizes its composition when the “flow-of-product” approach is used, see also Annex C.1. 70.5% of the GDP is attributed to personal consumption, 16.2% to investments, 18.4% to government consumption and -4.1% results from the net export. The latter means that the USA imported more goods and services than it exported.

Using the “earning or cost” approach – see also Annex C.1 – for the evaluation of the GDP, it is seen that 58.81% of the GDP is due to compensation of employees in terms of wages, salaries and supplements. 7.63% is due to proprietors' income, 1.61% to rental income, 7.63% to corporate profits, 5.63% are net interest, 6.68% refer to taxes, 0.92% are business current transfer payments and 0.01% correspond to current surplus of government enterprises. 12.54% are due to depreciation.

Figure B.3 illustrates the USA’s GDP increase over the last decades. It is obvious that the nominal GDP is larger than the real GDP. Considering the time period from 1965 to 2001, the nominal GDP increase may be described with an equivalent annual rate of 7.62%. The real GDP increased with an equivalent annual rate of 3.19%.
Appendix B – Detailed consequence assessment of the WTC failure

Table B.5 Composition of USA 2002 Gross Domestic Product, data: Bureau of Economic Analysis.

<table>
<thead>
<tr>
<th>Nominal GDP 2002 in billion USD</th>
<th>Flow of Products</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Gross domestic product</strong></td>
<td>10480.8 100.0%</td>
</tr>
<tr>
<td><strong>Personal consumption expenditures</strong></td>
<td>7385.3 70.5%</td>
</tr>
<tr>
<td>Durable goods</td>
<td>911.3 8.7%</td>
</tr>
<tr>
<td>Motor vehicles and parts</td>
<td>418.1 4.0%</td>
</tr>
<tr>
<td>Furniture and household equipment</td>
<td>323.7 3.1%</td>
</tr>
<tr>
<td>Other</td>
<td>169.4 1.6%</td>
</tr>
<tr>
<td><strong>Nondurable goods</strong></td>
<td></td>
</tr>
<tr>
<td>Food</td>
<td>1005.6 9.6%</td>
</tr>
<tr>
<td>Clothing and shoes</td>
<td>304.4 2.9%</td>
</tr>
<tr>
<td>Gasoline, fuel oil, and other energy goods</td>
<td>180.4 1.7%</td>
</tr>
<tr>
<td>Other</td>
<td>595.6 5.7%</td>
</tr>
<tr>
<td><strong>Services</strong></td>
<td></td>
</tr>
<tr>
<td>Housing</td>
<td>1144.6 10.9%</td>
</tr>
<tr>
<td>Household operation</td>
<td>408.2 3.9%</td>
</tr>
<tr>
<td>Electricity and gas</td>
<td>152.3 1.5%</td>
</tr>
<tr>
<td>Other household operation</td>
<td>255.9 2.4%</td>
</tr>
<tr>
<td>Transportation</td>
<td>292.8 2.8%</td>
</tr>
<tr>
<td>Medical care</td>
<td>1202.7 11.5%</td>
</tr>
<tr>
<td>Recreation</td>
<td>303.3 2.9%</td>
</tr>
<tr>
<td>Other</td>
<td>1036.4 9.9%</td>
</tr>
<tr>
<td><strong>Gross private domestic investment</strong></td>
<td>1589.2 15.2%</td>
</tr>
<tr>
<td>Fixed investment</td>
<td>1583.9 15.1%</td>
</tr>
<tr>
<td>Nonresidential</td>
<td>1080.2 10.3%</td>
</tr>
<tr>
<td>Structures</td>
<td>266.3 2.5%</td>
</tr>
<tr>
<td>Equipment and software</td>
<td>813.9 7.8%</td>
</tr>
<tr>
<td>Information processing equipment and software</td>
<td>421.3 4.0%</td>
</tr>
<tr>
<td>Computers and peripheral equipment</td>
<td>83.3 0.8%</td>
</tr>
<tr>
<td>Software</td>
<td>167.9 1.6%</td>
</tr>
<tr>
<td>Other</td>
<td>170.1 1.6%</td>
</tr>
<tr>
<td>Industrial equipment</td>
<td>137.5 1.3%</td>
</tr>
<tr>
<td>Transportation equipment</td>
<td>128.0 1.2%</td>
</tr>
<tr>
<td>Other equipment</td>
<td>127.1 1.2%</td>
</tr>
<tr>
<td>Residential</td>
<td>503.7 4.8%</td>
</tr>
<tr>
<td>Change in private inventories</td>
<td>5.4 0.1%</td>
</tr>
<tr>
<td>Farm</td>
<td>-3.4 0.0%</td>
</tr>
<tr>
<td>Nonfarm</td>
<td>8.7 0.1%</td>
</tr>
<tr>
<td><strong>Net exports of goods and services</strong></td>
<td>-426.3 -4.1%</td>
</tr>
<tr>
<td>Exports</td>
<td>1006.8 9.6%</td>
</tr>
<tr>
<td>Goods</td>
<td>697.8 6.7%</td>
</tr>
<tr>
<td>Services</td>
<td>309.1 2.9%</td>
</tr>
<tr>
<td>Imports</td>
<td>1433.1 13.7%</td>
</tr>
<tr>
<td>Goods</td>
<td>1190.3 11.4%</td>
</tr>
<tr>
<td>Services</td>
<td>242.7 2.3%</td>
</tr>
<tr>
<td><strong>Government consumption expenditure and gross investment</strong></td>
<td>1932.5 18.4%</td>
</tr>
<tr>
<td>Federal</td>
<td>679.5 6.5%</td>
</tr>
<tr>
<td>National defense</td>
<td>438.3 4.2%</td>
</tr>
<tr>
<td>Consumption expenditures</td>
<td>382.7 3.7%</td>
</tr>
<tr>
<td>Gross investment</td>
<td>55.7 0.5%</td>
</tr>
<tr>
<td>Nondefense</td>
<td>241.2 2.3%</td>
</tr>
<tr>
<td>Consumption expenditures</td>
<td>208.1 2.0%</td>
</tr>
<tr>
<td>Gross investment</td>
<td>33.0 0.3%</td>
</tr>
<tr>
<td>State and local</td>
<td>1253.1 12.0%</td>
</tr>
<tr>
<td>Consumption expenditures</td>
<td>1004.6 9.6%</td>
</tr>
<tr>
<td>Gross investment</td>
<td>248.4 2.4%</td>
</tr>
</tbody>
</table>
Influence on the economy

Figure B.2  United States of America’s age distribution, data: US Census 2000.

Figure B.3  USA’s Gross Domestic and NYS’s Gross State Product, data: Bureau of Economic Analysis.
B.3.1.2 New York State

Comparing the nominal GDP with the nominal gross state product (GSP) in Figure B.3, it is seen that in year 2000 the New York State’s GSP was around 8% of USA’s GDP. Between 1995 and 2001, this percentage varied between 8.04% and 8.21%. In 2001, the nominal GSP was 826.5 billion USD.

This gross state product was produced by the population of New York State (NYS), which in year 2000 was counted to 18’976’457. The territory of New York State comprises 7’679’307 housing units on 54’556 square miles (141’300 km²), of which 47’214 square miles (122’300 km²) are on land. This leads to an average density of 401.9 persons and 162.6 housing units per square mile (155 persons/km² and 62.8 housing units/km²), see Table B.6.

Figure B.4 shows the frequency distribution of the age of New York State’s population. This figure shows the same characteristics, which were observed in Figure B.2 for the age distribution of the USA’s population.

![Age distribution of New York State’s population, data: US Census 2000.](image)

B.3.1.3 New York City

Of the 19.0 million persons living in the state of New York, 8.0 million are living in New York City (NYC), which is 42.2%. New York City itself comprises five boroughs, namely Manhattan, Staten Island, Brooklyn, Queens and the Bronx. 30.9% of New York City’s population lives in Brooklyn (Kings County), 27.8% in Queens, 19.2% in Manhattan (New York County), 16.6% in the Bronx and 5.5% in Staten Island (Richmond County), see Table B.6. It is also seen that the population density and housing unit density is highest for Manhattan with 66’940 persons and 34’757 housing units per square mile (25’805 persons/km² and 13’400 housing units/km²). Figure B.5 shows the age distribution of the five boroughs’ population and the City of New York in total. It is seen that compared to the other boroughs, Bronx has a higher percentage of people younger than 20 years. Persons between 20 and 40 are especially attracted by Manhattan, whereas Staten Island shows a higher percentage of people from 40 to 60 and Queens has relatively more people between the age of 70 and 80. Brooklyn’s age distribution follows quite closely the average NYC age distribution.
Table B.6  


<table>
<thead>
<tr>
<th>Region</th>
<th>Population</th>
<th>Land Area</th>
<th>Housing Units</th>
<th>Population Density</th>
<th>Housing Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>New York State</td>
<td>18'976'457</td>
<td>47'214</td>
<td>7'679'307</td>
<td>402</td>
<td>163</td>
</tr>
<tr>
<td>New York City</td>
<td>8'008'278</td>
<td>303</td>
<td>3'200'912</td>
<td>26'402</td>
<td>10'553</td>
</tr>
<tr>
<td>Kings County</td>
<td>2'465'326</td>
<td>71</td>
<td>930'866</td>
<td>34'917</td>
<td>13'184</td>
</tr>
<tr>
<td>Bronx County</td>
<td>1'332'650</td>
<td>42</td>
<td>490'659</td>
<td>31'709</td>
<td>11'675</td>
</tr>
<tr>
<td>Queens County</td>
<td>2'229'379</td>
<td>109</td>
<td>817'250</td>
<td>20'409</td>
<td>7'482</td>
</tr>
<tr>
<td>New York County</td>
<td>1'537'195</td>
<td>23</td>
<td>798'144</td>
<td>66'940</td>
<td>34'757</td>
</tr>
<tr>
<td>Richmond County</td>
<td>443'728</td>
<td>58</td>
<td>163'393</td>
<td>7'588</td>
<td>2'804</td>
</tr>
</tbody>
</table>

Figure B.5  


Figure B.6 shows the development of NYC’s employment over more than five decades. A first peak can be observed in June 1969 and a second peak in December 1989. A last peak can be observed at December 2000, which marks the end of the boom induced by the information technology sector. At that time, 3.8 million people were employed in the so-called “nonfarm” sector. From this amount, 3.3 million person were employed by privat industry and 567’000 were working for federal, state or local governments. Considering the time series of the employees working in the public sector in more detail, it is seen that the events of September 11, 2001 did not significantly affect their employment.

Data from the New York State Department of Labor (NYSDoL) shows that around 54% of all units in New York City are located in Manhattan. Hereby, the NYSDoL defines a unit as “a single place of business, engaged in a single business activity, and operated by a single employer”. In the year 2000, Manhattan’s units employed 66% of NYC’s employees. They earned 81% of the total income generated in New York City.

For the year 2001, Table B.7 summarizes the total income earned by different sectors. The highest income is associated to employees in the finance and insurance industry, which in average earn 193’000 USD. This number also includes supplements, bonuses and taxes. The finance and insurance sector employs 13.9% of the employed people and they earn 35.9% of the total income. If in addition the information industry, professional and technical services and the government sector is considered, then 51.6% of the employed people and 68.8% of the total income is covered.
In 1998, 3.44 million people were employed in New York City producing a total income of 179 billion USD, of which 143 billion USD were attributed to Manhattan. In the same year, the City of New York produced a Gross City Product of 363.2 billion USD. Hence, the ratio of the GCP to the total income for 1998 is 2.03. In Figure B.8, the income and employment of Manhattan is shown for four decades.
Table B.7  Total income by different industries, data: NYSDoL.

<table>
<thead>
<tr>
<th>Industry</th>
<th>Annual Average Employment</th>
<th>Total Wages (in USD)</th>
<th>Annual Average Wage (in USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total, All Industries</td>
<td>2,341,467</td>
<td>175,358,194,800</td>
<td>74,892</td>
</tr>
<tr>
<td>Total, All Private</td>
<td>1,887,574</td>
<td>154,481,892,750</td>
<td>81,842</td>
</tr>
<tr>
<td>Agriculture, Forestry, Fishing &amp; Hunting</td>
<td>105</td>
<td>6,866,623</td>
<td>65,396</td>
</tr>
<tr>
<td>Mining</td>
<td>47</td>
<td>6,641,265</td>
<td>141,304</td>
</tr>
<tr>
<td>Construction</td>
<td>34,875</td>
<td>2,388,251,414</td>
<td>68,480</td>
</tr>
<tr>
<td>Manufacturing</td>
<td>60,656</td>
<td>2,955,818,489</td>
<td>48,731</td>
</tr>
<tr>
<td>Wholesale Trade</td>
<td>87,197</td>
<td>6,402,964,848</td>
<td>73,431</td>
</tr>
<tr>
<td>Retail Trade</td>
<td>127,601</td>
<td>4,602,977,318</td>
<td>36,073</td>
</tr>
<tr>
<td>Transportation and Warehousing</td>
<td>27,224</td>
<td>1,006,968,461</td>
<td>36,988</td>
</tr>
<tr>
<td>Information</td>
<td>157,632</td>
<td>13,260,514,341</td>
<td>84,123</td>
</tr>
<tr>
<td>Finance and Insurance</td>
<td>325,910</td>
<td>6,295,827,005</td>
<td>193,178</td>
</tr>
<tr>
<td>Real Estate and Rental and Leasing</td>
<td>74,590</td>
<td>4,136,690,275</td>
<td>55,459</td>
</tr>
<tr>
<td>Professional and Technical Services</td>
<td>271,246</td>
<td>23,604,724,809</td>
<td>87,023</td>
</tr>
<tr>
<td>Management of Companies and Enterprises</td>
<td>46,633</td>
<td>7,255,501,164</td>
<td>155,587</td>
</tr>
<tr>
<td>Administrative and Waste Services</td>
<td>144,918</td>
<td>5,629,461,810</td>
<td>38,846</td>
</tr>
<tr>
<td>Educational Services</td>
<td>70,341</td>
<td>2,902,546,451</td>
<td>41,264</td>
</tr>
<tr>
<td>Health Care and Social Assistance</td>
<td>185,538</td>
<td>7,668,566,170</td>
<td>41,332</td>
</tr>
<tr>
<td>Arts, Entertainment, and Recreation</td>
<td>44,510</td>
<td>2,353,793,378</td>
<td>52,882</td>
</tr>
<tr>
<td>Accommodation and Food Services</td>
<td>136,021</td>
<td>3,659,824,233</td>
<td>26,906</td>
</tr>
<tr>
<td>Other Services</td>
<td>82,261</td>
<td>3,011,672,829</td>
<td>36,611</td>
</tr>
<tr>
<td>Total, All Government</td>
<td>453,893</td>
<td>20,876,302,950</td>
<td>45,994</td>
</tr>
<tr>
<td>Unclassified</td>
<td>4,715</td>
<td>216,098,848</td>
<td>45,832</td>
</tr>
</tbody>
</table>

Figure B.8  Income and employment for Manhattan, data: NYSDoL.

B.3.1.4 Manhattan community district 1

Besides boroughs and counties, New York City is subdivided into community districts. There are 59 community districts in New York City, which have been established by local law in year 1975. In
size they range from less than 900 acres to almost 15,000 acres (3.5 – 60 km²), and in population they vary from less than 35,000 to more than 200,000.

Manhattan’s community district 1 is bounded by the East River in the east and the Hudson River in the west. In the north it is bounded by Canal, Baxter and Pearl Street and reaches down to the south tip of Manhattan. In addition, community district 1 includes Ellis, Liberty and Governors Island. Most of the population lives north and south of Brooklyn Bridge, in Battery Park City and in the northwest tip of community district 1.

Community district 1 comprises 1,100 acres of land area (4.45 km²) and in year 2000, 34,420 persons lived in that district. In Figure B.9, it is seen that around 30% of the Lower Manhattan’s area is covered with commercial and office buildings. In Figure B.10, the usage of Manhattan’s land for commercial activities is shown. Obviously, there are two economical centers in New York City, namely Downtown and Midtown Manhattan.

On September 11, 2001, the whole business activity in Manhattan was shut down. The area south of 14th Street remained closed for 3 days and the area south of Houston Street remained restricted for 5 days. One week after September 11, 2001, most of Lower Manhattan’s area was accessible to the public again, except for a restricted area which is bounded by the Hudson River in the west, Chambers Street in the north, Broadway in the east and Rector Street in the south. According to the World Trade Center Business Recovery Grant Program, businesses situated in that area were eligible to receive compensation for 25 days, see Figure B.11.

![Figure B.9](image-url)  
Manhattan’s community district 1, from New York Department of City Planning.
Influence on the economy

Figure B.10  Land use for commercial activities in Manhattan, from New York Department of City Planning.

Figure B.11  Days of economic loss following the Business Recovery Grant Program, from [141].

B.3.1.5 World Trade Center area

The World Trade Center was located in Downtown Manhattan in community district 1. The Twin Towers WTC1 and WTC2 alone provided almost 10 million sq ft office space, which was used by more than 34’000 employees. The whole complex comprised 14 million sq ft office and hotel space for more than 48’000 employees. The neighboring buildings, which were damaged by the failed towers, provided 21 million sq ft for more than 66’000 employees. In total this amounts to 141’124 employees on 35.4 million sq ft or 310 sq ft gross floor area per employee. The latter value represents office buildings but also considers among others the hotel in WTC3, the Verizon building,
which houses telecommunication facilities and the Fiterman Hall of the Borough of Manhattan Community College of the City University of New York, which was located in 30 West Broadway.

For WTC1, the employee density per gross floor area was 248 close to 250 (23.2 m²/employee), a number proposed by [39] and for WTC2 this value was 317 close to 300 (27.9 m²/employee) a number proposed by [117]. An upper bound for this density may be obtained when Figure B.12 is considered. This figure, which was used by eyewitnesses to describe the damage on the 91st floor of WTC1 [58], shows that an efficiently used floor may provide workspace for 229 employees at a density of 196 sq ft per employees (18.2 m²/employee). In particular, for office buildings older than the WTC this value seems to be an upper boundary because the new structural design of the WTC towers allowed 75% of the gross floor area to be rentable compared to 62% for older buildings [74].

Table B.8  WTC complex.

<table>
<thead>
<tr>
<th>Building</th>
<th>Size (sq ft)</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>WTC1</td>
<td>4'761'416</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC2</td>
<td>4'761'416</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC3</td>
<td>584'600</td>
<td>Hotel</td>
</tr>
<tr>
<td>WTC4</td>
<td>576'000</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC5</td>
<td>783'520</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC6</td>
<td>537'694</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC7</td>
<td>2'000'000</td>
<td>Class A Office</td>
</tr>
<tr>
<td>WTC Complex</td>
<td>14'004'646</td>
<td></td>
</tr>
</tbody>
</table>

Table B.9  Destroyed and damaged buildings, data: [124].

<table>
<thead>
<tr>
<th>Building</th>
<th>Space (sq ft)</th>
<th>Employees</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Destroyed Buildings:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WTC 1</td>
<td>4'761'416</td>
<td>19'175</td>
</tr>
<tr>
<td>WTC 2</td>
<td>4'761'416</td>
<td>14'997</td>
</tr>
<tr>
<td>WTC 3</td>
<td>584'600</td>
<td>1'249</td>
</tr>
<tr>
<td>WTC 4</td>
<td>576'000</td>
<td>1'861</td>
</tr>
<tr>
<td>WTC 5</td>
<td>783'520</td>
<td>3639</td>
</tr>
<tr>
<td>WTC 6</td>
<td>537'694</td>
<td>1'913</td>
</tr>
<tr>
<td>WTC 7</td>
<td>2'000'000</td>
<td>5245</td>
</tr>
<tr>
<td>Total Destroyed</td>
<td>14'004'645</td>
<td>48'079</td>
</tr>
</tbody>
</table>

| **Damaged Buildings:**    |               |           |
| WFC 1                     | 1'461'365     | 5'322     |
| WFC 2                     | 2'591'244     | 6'587     |
| WFC 3                     | 2'300'000     | 10'451    |
| WFC 4                     | 2'083'555     | 4'582     |
| Winter Garden              | 95'466        | 344       |
| 101 Barclay Street         | 1'226'000     | 4'414     |
| 111 Broadway               | 418'000       | 1'496     |
| 115 Broadway               | 399'000       | 1'422     |
| 140 Broadway               | 1'200'000     | 2'990     |
| 130 Cedar Street           | 135'000       | 485       |
| 90 Church Street           | 950'000       | 3'512     |
| 99 Church Street           | 336'000       | 383       |
| 100 Church Street          | 1'032'539     | 3'210     |
| 22 Cortlandt Street        | 668'110       | 3'037     |
| 24-26 Cortlandt Street     | 380'052       | 950       |
| 106 Liberty Street         | 18'000        | 64        |
| 110 Liberty Street         | 6'000         | 21        |
| 114 Liberty Street         | 69'000        | 246       |
| 1 Liberty Plaza            | 2'124'447     | 6'072     |
| 130 Liberty Plaza          | 1'415'086     | 5'051     |
| 75 Park Place              | 54'7'165      | 2'224     |
| 90 West Street             | 350'000       | 1'220     |
| 140 West Street            | 1'171'540     | 1'741     |
| 30 West Broadway           | 381'000       | 221       |
| Total Damaged              | 21'358'569    | 66'045    |

| Damaged and Destroyed      | 35'363'214    | 114'124   |
Influence on the economy

Figure B.12 91st floor of WTC1, from [58].

From the more than 114’000 employees that worked in the World Trade Center and in the surrounding buildings, finance, insurance and governmental enterprises represent 35% of the total enterprises; however, the enterprises employed 74% of the employees. The largest employer is the investment and finance sector, which with 63’300 employees accounts for 56% of the total employees.

In [124], 1’134 directly affected firms are listed, of which 311 stayed in Lower Manhattan. Over 192 moved to Midtown and 96 relocated to Manhattan Valley. In total New York City was able to retain 81% of their employment, see Figure B.14.

Figure B.13 Industries and employees of affected enterprises, data: [124].

Figure B.14 Relocation of affected businesses, from Wall Street Journal.
Figure B.15 compares the frequency distribution of the age between the WTC fatalities and the population of New York City. It is seen that the distributions differ significantly; however, based on the population’s age distribution an approximation for the age distribution of the WTC employees may be obtained. Such a distribution may then be used to model the consequences due to fatalities.

![Age frequency distribution of WTC fatalities, data: CNN, US Census 2000.](image)

### B.3.2 Business interruption

The present subsection considers the consequences due to business interruption related to the failure of the WTC Twin Towers. For this purpose, four studies are summarized, which aim to assess the socio-economical consequences of this event. Two of the reports were published by the Comptroller of New York City. The first report was published on October 4, 2001 [117] and the second on September 4, 2002 [118]. Furthermore, the New York City Partnership and Chamber of Commerce published a report with authors representing expertise of renowned consulting firms [122]. Finally in November 2002, the Federal Reserve Bank of New York published a report in order to assess the economical impact of the September 11, 2001, see [70].

#### B.3.2.1 Comptroller of New York City, Oct. 2001

Already in October 4, 2001 i.e. less than a month after the failure of the WTC Twin Towers, the office of the Comptroller of New York City published the first assessment of the socio-economical effects of the failure of the WTC Twin Towers [117]. Table B.10 summarizes the assessed socio-economic consequences, which according to the report, range from 90 to 105 billion USD.

In order to assess business interruption losses, the Comptroller of New York City calculates the lost added value of the gross city product (GCP). The report assumes that 75% of the GCP is produced in Manhattan, of which one third is generated in Lower Manhattan. Accordingly, one quarter of the GCP is produced in Lower Manhattan. Furthermore, the report states that New York City’s GCP is 373 billion USD, which averages to 7 billion USD per week. Moreover, the comptroller’s report estimates that 1.2 billion USD is produced at business days and 1.0 billion USD at weekends. The above mentioned contribution of Manhattan and Downtown New York to the GCP results in 0.9 billion USD per business day for Manhattan and 300 million USD for Downtown New York as the region’s added value to the GCP.
Table B.10  Socio-economic consequences according to [117].

<table>
<thead>
<tr>
<th>Item</th>
<th>Loss Estimate</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td>Property</td>
<td>34</td>
<td>34</td>
</tr>
<tr>
<td>Cleanup/Rescue/Security</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>Fatalities</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>Human Disability/Trauma</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Business Interruption/Training/Unemployment</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>Lost Rents from Jobs Relocated Out of NYC</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Lost Wages from Jobs Relocated Out of NYC</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Long Term Lost Rents and Wages</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td><strong>Total Loss</strong></td>
<td><strong>90</strong></td>
<td><strong>105</strong></td>
</tr>
</tbody>
</table>

*(in billion USD)*

Based on this and further estimates for the percentage loss, the comptroller of New York City assesses the loss from business interruption. For the 10 month period after September 11, 2001, it estimates the business interruption loss to 20.9 billion USD, see Table B.11.

Table B.11  Business interruption losses according to [117].

<table>
<thead>
<tr>
<th>Period</th>
<th>Location</th>
<th>GCP/period</th>
<th>Percent</th>
<th>Loss</th>
<th>Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>First 4 days</td>
<td>Lower Manhattan</td>
<td>0.3 billion USD/day</td>
<td>90.0%</td>
<td>1.08 billion USD</td>
<td></td>
</tr>
<tr>
<td>First 4 days</td>
<td>Rest of NYC</td>
<td>0.9 billion USD/day</td>
<td>20.0%</td>
<td>0.72 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 4 weeks</td>
<td>Lower Manhattan</td>
<td>1.7 billion USD/week</td>
<td>25.0%</td>
<td>1.70 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 4 weeks</td>
<td>Rest of NYC</td>
<td>5.3 billion USD/week</td>
<td>15.0%</td>
<td>3.18 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 9 months</td>
<td>Lower Manhattan</td>
<td>7.5 billion USD/month</td>
<td>10.0%</td>
<td>6.75 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 9 months</td>
<td>Rest of NYC</td>
<td>15.0 billion USD/month</td>
<td>5.5%</td>
<td>7.42 billion USD</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>20.85 billion USD</strong></td>
<td></td>
</tr>
</tbody>
</table>

From the 373.3 billion USD of GCP, 169.8 billion USD are associated to wages. Hence, with a 45.5% share of the GCP, the lost wages are estimated to 9.58 billion USD, which corresponds to a loss of 115 thousand full-time employees. Table B.12 summarizes the lost added value, jobs and wages by industry.

Table B.12  Business interruption, job and wage losses by industry according to [117].

<table>
<thead>
<tr>
<th>Industry</th>
<th>Lost GCP mill. USD</th>
<th>Total Wages mill. USD</th>
<th>Employees thousands</th>
<th>Wage Rate USD</th>
<th>Lost Jobs</th>
<th>Lost Wages USD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Street</td>
<td>7'500</td>
<td>37'325</td>
<td>163</td>
<td>229'499</td>
<td>14'865</td>
<td>3'411</td>
</tr>
<tr>
<td>Banking</td>
<td>1'500</td>
<td>8'112</td>
<td>82</td>
<td>98'928</td>
<td>6'697</td>
<td>682</td>
</tr>
<tr>
<td>Insurance</td>
<td>1'300</td>
<td>6'430</td>
<td>65</td>
<td>98'928</td>
<td>5'977</td>
<td>591</td>
</tr>
<tr>
<td>Real Estate</td>
<td>1'000</td>
<td>10'486</td>
<td>106</td>
<td>98'928</td>
<td>4'598</td>
<td>455</td>
</tr>
<tr>
<td>Trade</td>
<td>1'700</td>
<td>15'232</td>
<td>345</td>
<td>44'215</td>
<td>17'489</td>
<td>773</td>
</tr>
<tr>
<td>Restaurants</td>
<td>1'000</td>
<td>6'652</td>
<td>120</td>
<td>55'439</td>
<td>8'205</td>
<td>455</td>
</tr>
<tr>
<td>Theater</td>
<td>500</td>
<td>2'218</td>
<td>40</td>
<td>55'439</td>
<td>4'102</td>
<td>227</td>
</tr>
<tr>
<td>Hotels</td>
<td>750</td>
<td>1'940</td>
<td>35</td>
<td>55'439</td>
<td>6'154</td>
<td>341</td>
</tr>
<tr>
<td>Legal</td>
<td>300</td>
<td>4'435</td>
<td>80</td>
<td>55'439</td>
<td>2'461</td>
<td>136</td>
</tr>
<tr>
<td>Other Service</td>
<td>3'500</td>
<td>34'591</td>
<td>624</td>
<td>55'439</td>
<td>28'717</td>
<td>1'592</td>
</tr>
<tr>
<td>Other</td>
<td>2'000</td>
<td>42'006</td>
<td>730</td>
<td>57'543</td>
<td>15'809</td>
<td>910</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>21'050</td>
<td>169'429</td>
<td>2'389</td>
<td>115'274</td>
<td>9'575</td>
<td></td>
</tr>
</tbody>
</table>

B.3.2.2  Comptroller of New York City, Sept. 2002

One year later, on September 4, 2002 the comptroller of New York City published an update of its socio-economic impacts study [118]. According to this report, the total loss varies from 82.8 to 94.8 billion USD, i.e. roughly 10% less than the previous estimate. Comparing Table B.13 with Table B.10, it is obvious that the costs associated to property losses and clean-up have been updated as new
information became available. They were reduced from 48 billion USD (34 plus 14) to 21.8 billion USD. Consequences due to fatalities were reduced from 14 billion USD (3 plus 11) to 8.7 billion USD. However, the estimated economic losses increased from 27 – 43 to 53.3 – 64.3 billion USD. One reason for this is that the considered period for which consequences are assessed has been extended from 10 month to 3.25 years.

Table B.13 Consequences according to [118].

<table>
<thead>
<tr>
<th>Item</th>
<th>Subtotals</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
<td>21.8</td>
<td></td>
</tr>
<tr>
<td>Fatalities</td>
<td>8.7</td>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td><strong>30.5</strong></td>
<td></td>
</tr>
<tr>
<td>Lost Gross City Product: 2001 (3 months)</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>Lost Gross City Product: 2002</td>
<td>15.8</td>
<td></td>
</tr>
<tr>
<td>Lost Gross City Product: 2003-2004</td>
<td>25 - 37</td>
<td></td>
</tr>
<tr>
<td><strong>Lost Gross City Product: 2001-04</strong></td>
<td><strong>52.3 - 64.3</strong></td>
<td></td>
</tr>
</tbody>
</table>

Total Economic Impact 82.8 - 94.8

Based on the same methodology as in the previous report and some updated values, the Comptroller estimates an economic loss of 11.5 billion USD for the remaining year 2001, see Table B.14.

Table B.14 Business interruption losses for remaining year 2001 according to [118].

<table>
<thead>
<tr>
<th>Period</th>
<th>Location</th>
<th>GCP/period</th>
<th>Percent</th>
<th>Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>First 4 days Lower Manhattan</td>
<td>0.3 billion USD/day</td>
<td>90%</td>
<td>1.08 billion USD</td>
<td></td>
</tr>
<tr>
<td>First 4 days Rest of NYC</td>
<td>0.9 billion USD/day</td>
<td>20%</td>
<td>0.18 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 4 weeks Lower Manhattan</td>
<td>2.1 billion USD/week</td>
<td>30%</td>
<td>0.63 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 4 weeks Rest of NYC</td>
<td>6.3 billion USD/week</td>
<td>15%</td>
<td>0.945 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 10 weeks Lower Manhattan</td>
<td>2.1 billion USD/week</td>
<td>10%</td>
<td>0.21 billion USD</td>
<td></td>
</tr>
<tr>
<td>Next 10 weeks Rest of NYC</td>
<td>6.3 billion USD/week</td>
<td>2%</td>
<td>0.126 billion USD</td>
<td></td>
</tr>
</tbody>
</table>

**Total 2001:** 11.46 billion USD

For the year 2002, the Comptroller of New York City calculates a Gross City Product of 430 billion USD. This value is 19.8 billion USD short of the value, which was projected before September 2001, see Figure B.16. Between 2002 and 2004 this amounts to 76.4 billion in 1996 USD. Multiplying this value with a factor of 1.1, the comptroller obtains the nominal loss for 2001 of 84.0 billion USD, see Annex C.3. Furthermore, the report questions whether or not the full sum should be attributed to the events of September 11. Due to economic cycles, the economy went into a recession, the stock market dropped and consumer and investor confidence was lost from accounting irregularities of corporations such as Enron. The comptroller attributes more than 50% of the loss to September 11, 2001. Finally, it assesses the total economical loss to be between 52.3 and 64.3 billion USD.

Figure B.16 Development of NYC’s real Gross City Product in 1996 USD, from [118].
B.3.2.3 New York City Partnership

In November 2001, the New York City Partnership and Chamber of Commerce published a report analyzing the economic impact, which resulted from the failure of the World Trade Center Twin Towers [122]. The authors of the report are experts from renowned consulting firms, namely A.T. Kearny, Bain & Co., the Boston Consulting Group, Booz-Allen & Hamilton, KPMG, McKinsey & Co. and PricewaterhouseCoopers.

The report estimates that in the fourth quarter of 2001, 125,000 jobs were lost and that from this amount 57,000 persons were still unemployed by the end of 2003. Furthermore, the report summarizes the total socio-economic effect to 83 billion USD. This comprises 14 billion USD in clean-up costs, which includes debris removal and security expenses. Moreover, property costs of 29.7 billion USD are included as well as economic losses of 27.3 billion USD. The latter is multiplied by 1.42 so that 38.8 billion USD is obtained. The factor 1.42 considers the multiplier effect explained in Annex C.2. Moreover, the report estimates that 37 billion USD will be received in insurance payments, which will have an effect of 47 billion USD due to multiplier effects. Furthermore, this report estimates reimbursements of 14 billion USD from the federal government with an effect of 20 billion USD from multiplier effects. Finally, the report assesses a net socio-economic impact onto New York City of 16 billion USD.

Table B.14 summarizes the economic impact according to New York City’s industrial sectors. In this table, the economic impact is differentiated into “short term”, the period from September 11 to December 31, 2001 and “log term”, the two year period from January 2002 to December 2003.

Table B.14 Economic loss by industry, data: [122]

<table>
<thead>
<tr>
<th>Industry Sector</th>
<th>Short Term</th>
<th>Long Term</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Financial Services</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
<tr>
<td>Tourism / Transport</td>
<td>0.5</td>
<td>3.1</td>
<td>3.6</td>
</tr>
<tr>
<td>Retail</td>
<td>0.7</td>
<td>2.9</td>
<td>3.6</td>
</tr>
<tr>
<td>Manufacturing</td>
<td>0.5</td>
<td>1.9</td>
<td>2.4</td>
</tr>
<tr>
<td>Wholesale Trade</td>
<td>0.3</td>
<td>1.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Professional Services</td>
<td>0.8</td>
<td>0.1</td>
<td>0.9</td>
</tr>
<tr>
<td>Media / Entertainment</td>
<td>0.4</td>
<td>0.5</td>
<td>0.9</td>
</tr>
<tr>
<td>IT / Communications</td>
<td>0.3</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td>Government</td>
<td>0.7</td>
<td>0.1</td>
<td>0.8</td>
</tr>
<tr>
<td>Health Services</td>
<td>0.2</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>Insurance</td>
<td>0.1</td>
<td>0.0</td>
<td>0.1</td>
</tr>
<tr>
<td>Energy</td>
<td>0.0</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Real Estate / Construction</td>
<td>0.0</td>
<td>-1.3</td>
<td>-1.3</td>
</tr>
<tr>
<td>Other</td>
<td>0.5</td>
<td>0.0</td>
<td>0.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7.5</strong></td>
<td><strong>19.8</strong></td>
<td><strong>27.3</strong></td>
</tr>
</tbody>
</table>

Finally, the report assigns almost half of the economic loss to the financial service sector, which sustained an economic loss of 12.7 billion USD. The total economic loss is assessed to 27.3 billion USD, whereby the report applies a multiplier of 1.42 to obtain an economic loss including multiplier effects of 38.8 billion USD. In Table B.14, it is indicated that the influence of the failure of the WTC Twin Towers on the real estate and construction sector results in negative
Appendix B – Detailed consequence assessment of the WTC failure

costs, which means a positive effect of 1.3 billion USD. The report gives no further explanation about that number; however, it may result from the effect of the reconstruction activity in Lower Manhattan.

B.3.2.4 Federal Reserve Bank of New York, Nov. 2002

In November 2002, the Federal Reserve Bank of New York published a report in order to measure the economic impact of September 11, 2001 [70]. This report, estimates the total socio-economic consequences to lie between 33.0 and 35.8 billion USD. It considers the costs for clean-up and property replacement as the largest contributor, which amount to 21.6 billion USD. The fatalities’ earnings are assessed to 7.8 billion USD and economic losses are measured with the reduced wages and salaries, which range from 3.6 to 6.4 billion USD, see Table B.15.

<table>
<thead>
<tr>
<th>Item</th>
<th>Loss Estimate (in billion USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property &amp; Cleanup</td>
<td>21.6</td>
</tr>
<tr>
<td>Fatalities Earning</td>
<td>7.8</td>
</tr>
<tr>
<td>Lost Income</td>
<td>3.6</td>
</tr>
<tr>
<td>Total</td>
<td>33.0</td>
</tr>
</tbody>
</table>

The report states that in the period from 1989 to 1992 New York City suffered an economic recession, during which 344’000 jobs were lost; this corresponds to 11.4%. In the period from 1996 to 2000, New York City experienced its strongest economical boom of the past half century [71]. The private sector employment grew with an average annual rate of 2.6% and at the same time, wages and salaries grew at an average annual rate of 9.6% or 7%, when adjusted for inflation. Wall Street earnings in particular increased with an inflation adjusted average annual rate of 16%. Next to the securities industry, the advertising, motion picture, publishing, media, tourism, business and computer services experienced a business expansion, which cannot entirely be attributed to a multiplier effect of Wall Street. The Employment in New York City reached a maximum in December 2000. After that it declined as an effect of an economical downward cycle. From December 2000 to March 2002, the private sector employment was reduced by 147’000 employees or 4.7%. The report indicates that 55’000 jobs were lost in the period from January to September 2001, which is more than one third. Then in October, 51’000 jobs were lost additionally. In that month, the financial sector alone lost 12’000 jobs and the catering industry lost 9’000 jobs. The air transportation industry lost 11’000 jobs, almost all in the period of October and November 2001. In addition, hotels laid off 6’000 persons between September 2001 and March 2002.

In order to assess the lost jobs due to the failure of the WTC Twin Towers, the Federal Reserve Bank developed an autoregressive forecasting model. This model includes a relationship between employment growth in New York City and the rest of the USA. It estimates the employment development on a pre-September 2001 basis. The lost jobs are then obtained as the difference of the forecasted and the actual number. For the calculation of the employment situation, autoregressive models of order 3 to 8 were fitted. The lowest employment and therefore the lowest impact scenario is obtained using order 8 because this process has a long memory and the recession has a longer sustaining influence. The highest employment is calculated with order 3 and represents the high impact scenario.

For the low impact scenario, 38’000 lost jobs are calculated for October 2001, 49’000 for February and 28’000 persons were still unemployed in June 2002. With an average annual earning of 115’470 USD, the lost wages and salaries are obtained to 3.6 billion USD. The high impact scenario implies 46’000 lost jobs in October 2001 and 71’000 in February 2002; 55’000 are unemployed due to September 11 in June 2002. Furthermore, a higher average annual income of 141’821 USD is assumed resulting in a lost income of 6.4 billion USD. It is noted that these values only reflect the lost income experienced by the persons who were laid-off. However the values do not consider lost corporate profits, tax revenues, etc. When this is included, the loss is around twice as much.

In [71], additional calculations were performed by comparing the actual economy with a statistical simulation of how New York City’s economy would have performed if the World Trade Center Twin
Influence on the economy

Towers did not fail. These calculations show that the unemployment due to September 11 reached a peak in February 2002 and then declined gradually in the subsequent months. In addition, the report states that the effects on the city’s economy were somewhat less severe than first estimated because the downturn to a large degree stems from the economic recession.

B.3.2.5 Summary

Table B.16 summarizes the estimated economic losses from the four reports. The first three reports assess the economic loss by means of the lost gross city product. The Federal Reserve Bank, however, considers only the lost income, whereas the lost GCP is around twice as much. In order to identify the lost GCP, the first and second report estimate the GCP, which is produced in a specific region and then assess its lost percentage share and period. The three other reports calculate the difference of the projected and the actual realized GCP. Considering Table B.16, in which the lost income identified by the Federal Reserve Bank was multiplied with the gross city product to income ratio (2.0) to obtain the lost GCP, it is seen that the largest value is almost nine times as much as the smallest value. This shows the inherent difficulties in the assessment of economic losses and the associated uncertainties. Firstly, the economical development of an undisturbed economy may be modeled in different ways, e.g. by using the theory of stochastic processes. Secondly, the differentiation of economic losses into losses induced by the events of September 11 and losses resulting from recession is made subjectively according to expert judgment. Thirdly, the reports also vary in the period they assume that the economy is affected from the September 11 events, see also [73]. Furthermore, in [70] only economic losses which are associated with layoffs are considered. However, enterprises also experience losses without laying off its employees immediately.

In addition to the reports summarized in the foregoing [117], [118], [122] and [70], the following reports analyze the economic impact of September 11, 2001, as well: [106], [39], [127], [67] and [68]. Generally, they use the same methodology for the assessment of economic consequences and some are somewhat less comprehensive.

Table B.16 Economic losses from September 11.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Economic losses</td>
<td>28.0</td>
<td>43.0</td>
<td>52.3</td>
<td>64.3</td>
<td>38.8</td>
<td>38.8</td>
<td>3.6</td>
<td>6.4</td>
</tr>
</tbody>
</table>

B.3.3 Economical losses of infrastructural facilities

B.3.3.1 Metropolitan Transportation Authority (MTA)

The New York State’s Comptroller estimated the lost revenue of the Metropolitan Transportation Authority (MTA) in December 2001, whereby the loss is calculated for a 15 month period, see Table B.17. The total lost revenue and higher expenses were estimated to 531 million USD. Almost half of this amount, 244.6 million USD, was attributed to lost revenues from bus, subway and railroad fares and bridge and tunnel tolls. From this amount, 99.6 million USD accounts for lost revenue associated with New York City Transit, 89.5 million USD comes from bridge and tunnel closures. The Long Island and Metro-North railroads have lost revenues of 38.7 and 16.8 million USD, respectively.

The amount of 123.4 million USD in losses is due to debris removal, security and structural assessments. 163 million USD are attributed to lost tax revenues. For the calendar years 2001 and 2002, the total deficit is calculated to 264 million USD, of which however, 101 million USD are due to the economical recession and 163 million USD are due to the failure of the WTC Twin Towers [128]. It is noted that the lost tax revenues are already considered by business interruption when the lost GCP or GDP is calculated. In [104], an updated number for the lost revenue and additional expenses is indicated with 482 million USD.
Appendix B – Detailed consequence assessment of the WTC failure

Table B.17  MTA’s economic losses according to [128].

<table>
<thead>
<tr>
<th></th>
<th>Operating Revenues</th>
<th>Tax Revenues</th>
<th>Expenses</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>MTA Headquarters</td>
<td>0.0</td>
<td>31.9</td>
<td>20.6</td>
<td>52.5</td>
</tr>
<tr>
<td>NYC Transit</td>
<td>99.6</td>
<td>130.7</td>
<td>95.1</td>
<td>325.4</td>
</tr>
<tr>
<td>TBTA</td>
<td>89.5</td>
<td>0.0</td>
<td>7.0</td>
<td>96.5</td>
</tr>
<tr>
<td>LIRR</td>
<td>38.7</td>
<td>0.0</td>
<td>0.5</td>
<td>39.2</td>
</tr>
<tr>
<td>Metro-North</td>
<td>16.8</td>
<td>0.0</td>
<td>0.1</td>
<td>16.9</td>
</tr>
<tr>
<td>LI Bus</td>
<td>0.0</td>
<td>0.0</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>244.6</strong></td>
<td><strong>162.6</strong></td>
<td><strong>123.4</strong></td>
<td><strong>530.6</strong></td>
</tr>
</tbody>
</table>

(in million USD)

B.3.3.2 Port Authority of New York and New Jersey

In [131], the losses of the Port Authority of New York and New Jersey are summarized, see Table B.18. The table shows that the lost revenue and additional expenses associated with the failure of the WTC Twin Towers are estimated to 259 million USD. The restoration of the ferry service and its temporary expansion did cost 100 million USD. The lost revenue from the airports is estimated to 35 million USD, the maritime port lost 10 million USD in revenues; 35 million USD are due to reduced automobile traffic, e.g. due to the closure of the Holland Tunnel. The temporary closure of the bus terminal and the reduced schedule cost 9 million USD in lost ticket and retail sales. Finally, the reduced use of the PATH trains costs 70 million in lost revenues. Property losses are estimated to 1.04 billion USD and additional security expenses for the Port Authority’s facilities are estimated to 1.15 billion USD. Whereas, property losses have been assessed in Annex B.2.2.5, the security expenses are not followed up in this report because such costs would not have occurred had the cause of the WTC failure not been malicious.

Table B.18  Port Authority’s loss estimates.

<table>
<thead>
<tr>
<th>Loss</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>PATH</td>
<td>850</td>
</tr>
<tr>
<td>Headquater</td>
<td>150</td>
</tr>
<tr>
<td>Powerstation</td>
<td>40</td>
</tr>
<tr>
<td><strong>Material Loss</strong></td>
<td>1'040</td>
</tr>
<tr>
<td>Ferry</td>
<td>100</td>
</tr>
<tr>
<td>Airports</td>
<td>35</td>
</tr>
<tr>
<td>Marine Ports</td>
<td>10</td>
</tr>
<tr>
<td>Car Traffic</td>
<td>35</td>
</tr>
<tr>
<td>Bus Terminal</td>
<td>9</td>
</tr>
<tr>
<td>PATH</td>
<td>70</td>
</tr>
<tr>
<td><strong>Revenue Loss &amp; Additional Expenses</strong></td>
<td><strong>259</strong></td>
</tr>
<tr>
<td>Airport Security</td>
<td>960</td>
</tr>
<tr>
<td>Maritime Port, Bridges &amp; Tunnel Security</td>
<td>190</td>
</tr>
<tr>
<td><strong>Security Expenses</strong></td>
<td><strong>1'150</strong></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>2'449</strong></td>
</tr>
</tbody>
</table>

(in million USD)

B.3.4 Lost rents

Generally, costs for rents are already considered indirectly in the economic losses, which have been assessed by means of the GCP or GDP because the GDP includes rents, when the cost approach is used, see also Appendix C. However, the considered time period to assess the economic impact and the lost rents may be different. Moreover, lost rents may become a significant consequence, especially when structures have to be reconstructed and when the considered building is situated in the city, which is the world’s fourth most expensive office location and the city with the world highest retail rents, [35] and [34]. At Fifth Avenue retail space is as expensive as 9’149 USD per square meter per year.
With the 13.4 million sq ft of office space, the WTC – excluding WTC3, the Marriot hotel – contained less than 4% of Manhattan’s entire office space. In 2001, Manhattans office space was estimated to 353.7 million sq ft [83]. In total 37.5 million sq ft office space, which is more than 30% of downtown’s office space, was either destroyed or damaged and could not be rented. 27.8 million sq ft of it was Class A office space; this represented 60% of Downtown’s Class A office space [186]. Figure B.18 shows the development of Manhattans office market by means of the two real estate sector’s most important indicators, i.e. the vacancy rate and the average asking rent. Comparing Figure B.18 with Figure B.19, it is obvious that there is a relation between the employment situation and the vacancy rate, as well as between the vacancy rate and that the average asking rate. After the loss of more than 30 million sq ft of Manhattan’s office space, it was expected that the vacancy rate would decline due to the need for replacement space and the average asking rent would rise. However, the opposite occurred: the vacancy rate rose and the average rent declined, since the demand weakened more than it was assumed. In addition, there was enough extra space available for affected companies. For instance, some hotels were happy to replace beds with desks for temporary office space, as the market for business travels and tourism declined [70].

Figure B.18 Manhattan’s office market, vacancy rate and average asking rent, from [16].

Figure B.19 NYC Employment, data: Bureau of Labor Statistic.
B.3.4.1 The WTC leases

On July 24, 2001 the lease contract between the Port Authority and the two enterprises Silverstein Properties Inc. and Westfield America Inc. started [154]. The lease comprises the two 110 storey twin towers WTC1 and WTC2 as well as the nine storey buildings WTC4 and WTC5. After a 616 million upon commencement payment, annual rents have to be paid starting at 102 million USD. The nominal value of the 99 year lease is indicated with 97.6 billion USD, which corresponds to a net present value of 3.22 billion USD. The lease contract establishes both a concomitant right and an obligation for the lessees to reconstruct the leased buildings after a failure, at their own costs and expenses, whether or not the damage is covered by insurance payment, see [154].

In 2001, WTC3 was under a 99 year lease as well, which commenced in 1995 with Host Marriot Corporation. This lease imposes a general obligation on the net lessee to restore the building following a failure or collapse. However, the Port Authority is obliged to restore the building’s support columns and the means to access the facility.

The lease of WTC7 was signed in 1980 between the Port Authority and the 7 World Trade Company, an enterprise affiliated to Silverstein Properties, Inc. The initial term is 39 years with 3 renewal options.

In 1970, WTC6 was leased to the United States of America with an initial term of 20 years and provision for 16 renewal options of five years. This lease obligates the Port Authority to rebuild the structure with no restoration or insurance obligation upon the lessee. Furthermore, the lessee is entitled to rent abatements. The rent for WTC7 has been reduced to the prescribed minimum rent, whereas the leases for WTC1, 2, 3, 4 and WTC5 do not provide for rent abatement before or during the restoration period.

B.3.4.2 Summarizing lost rents

Table B.19 summarizes the revenues, which the Port Authority obtained by the operation of the World Trade Center from 2000 to 2002. It is noted that the budget for 2001 assumed a full year operation of the World Trade Center under the Port Authority’s control, whereas the budget year 2002 reflects the commenced lease. It is seen that the income from operation comprise the gross operating revenue minus the operation and maintenance costs, the administration cost and depreciation. For the considered time period, the income from operation is increasing and is in the order of 100 million USD per annum. When furthermore, interests are subtracted then the net income is obtained.

Table B.19 Revenue from WTC according to [154].

<table>
<thead>
<tr>
<th>WTC</th>
<th>2000</th>
<th>2001</th>
<th>2002</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross operating revenues</td>
<td>347'659</td>
<td>291'439</td>
<td>129'670</td>
</tr>
<tr>
<td>Operating and maintenance</td>
<td>194'624</td>
<td>149'994</td>
<td>17'036</td>
</tr>
<tr>
<td>General administration</td>
<td>9'287</td>
<td>8'169</td>
<td>4'965</td>
</tr>
<tr>
<td>Depreciation</td>
<td>52'182</td>
<td>37'971</td>
<td>0</td>
</tr>
<tr>
<td><strong>Income from operation</strong></td>
<td><strong>91'566</strong></td>
<td><strong>95'305</strong></td>
<td><strong>107'669</strong></td>
</tr>
<tr>
<td>Interests</td>
<td>58'336</td>
<td>36'767</td>
<td>29'870</td>
</tr>
<tr>
<td><strong>Net income</strong></td>
<td><strong>33'230</strong></td>
<td><strong>58'538</strong></td>
<td><strong>77'799</strong></td>
</tr>
</tbody>
</table>

According to [142], the World Trade Center complex will be fully developed in the end of 2013. Based on an annual loss equal to the annual income from operation, because interests have to be paid, while the other costs as well as the revenue will not occur, the lost rent for the World Trade Center is estimated to 1.20 billion USD. Hereby, it is assumed that the present value of the income from the operation is constant. This estimate does not consider that WTC7 is expected to be completed late in 2005 or early in 2006 [136]. The lost rents of the surrounding buildings are also not considered. Most of these buildings were re-opened before April 2002, see Table B.20. Hence, lost rents can roughly be estimated to 1.2 billion USD.
Fatalities

Table B.20 Status of surrounding buildings, data: [114].

<table>
<thead>
<tr>
<th>No.</th>
<th>Building</th>
<th>Size</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>World Financial Center</td>
<td>1'461'365</td>
<td>Re-opened November 2001</td>
</tr>
<tr>
<td>2</td>
<td>World Financial Center</td>
<td>2'591'244</td>
<td>Re-opened April 2002</td>
</tr>
<tr>
<td>3</td>
<td>World Financial Center</td>
<td>2'083'555</td>
<td>Re-opened April 2002</td>
</tr>
<tr>
<td>4</td>
<td>World Financial Center</td>
<td>2'083'555</td>
<td>Re-opened December 2001</td>
</tr>
<tr>
<td>5</td>
<td>West Street</td>
<td>1'171'540</td>
<td>Re-opening Winter 2003-2004</td>
</tr>
<tr>
<td>6</td>
<td>West Broadway</td>
<td>381'090</td>
<td>Insurance Claim filed</td>
</tr>
<tr>
<td>7</td>
<td>Church Street</td>
<td>950'000</td>
<td>Re-opening Spring 2004</td>
</tr>
<tr>
<td>8</td>
<td>Broadway</td>
<td>875'000</td>
<td>Re-opened</td>
</tr>
<tr>
<td>9</td>
<td>Cortland Street</td>
<td>668'110</td>
<td>Re-opened March 2002</td>
</tr>
<tr>
<td>10</td>
<td>Cortland Street</td>
<td>25'000</td>
<td>Re-opened</td>
</tr>
<tr>
<td>11</td>
<td>Liberty Plaza</td>
<td>2'121'437</td>
<td>Re-opened October 2001</td>
</tr>
<tr>
<td>12</td>
<td>Liberty Street</td>
<td>18'000</td>
<td>Re-opened</td>
</tr>
<tr>
<td>13</td>
<td>Liberty Street</td>
<td>6'000</td>
<td>Re-opened</td>
</tr>
<tr>
<td>14</td>
<td>Liberty Street</td>
<td>1'415'086</td>
<td>Appointed for Demolition</td>
</tr>
<tr>
<td>15</td>
<td>Cedar Street</td>
<td>135'000</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

Table B.21 Lost rents according to Comptroller of New York City [117].

<table>
<thead>
<tr>
<th>Building</th>
<th>Destroyed</th>
<th>Damaged</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>WTC Towers</td>
<td>9'523</td>
<td>0</td>
<td>9'523</td>
</tr>
<tr>
<td>Rest of WTC Complex</td>
<td>3'897</td>
<td>0</td>
<td>3'897</td>
</tr>
<tr>
<td>Surrounding Damaged Office Buildings</td>
<td>0</td>
<td>16'583</td>
<td>16'583</td>
</tr>
<tr>
<td>Nearby Buildings Closed</td>
<td>0</td>
<td>8'500</td>
<td>8'500</td>
</tr>
<tr>
<td><strong>Affected area in 1'000 sq ft</strong></td>
<td>13'420</td>
<td>25'083</td>
<td>38'503</td>
</tr>
<tr>
<td>Percentage Loss</td>
<td>100%</td>
<td>40%</td>
<td></td>
</tr>
<tr>
<td>Average Annual Rent USD per sq.ft.</td>
<td>75.00</td>
<td>75.00</td>
<td>75.00</td>
</tr>
<tr>
<td><strong>Lost Rent in billion USD</strong></td>
<td>1.00</td>
<td>0.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Relocated in NYC, bil USD</td>
<td>0.50</td>
<td>0.20</td>
<td>0.70</td>
</tr>
<tr>
<td><strong>Net Lost Rent in billion USD</strong></td>
<td>0.50</td>
<td>0.55</td>
<td>1.05</td>
</tr>
</tbody>
</table>

In [117] the Comptroller of New York City estimates the lost rents for a one year period to 1.00 billion USD from destroyed buildings and 0.75 billion from damaged buildings. It is assumed that 0.70 billion USD would be spent for rents at other places in New York City, so that a net loss of 1.05 billion USD is obtained. The calculations are based on the gross rent, which was assumed as high as 75 USD per sq ft per year. The percentage loss of the damaged buildings was assumed to 40%, which corresponds to a downtime of 5 month, see also Table B.21.

B.3.5 Summary

Due to the failure of the WTC Twin Towers the economy was negatively affected. The loss due to business interruption is estimated between 7.2 and 64.3 billion USD. Furthermore, the Metropolitan Transportation Authority experienced lost revenues and additional expenses of 482 million USD. In addition, the Port Authority lost revenues and experienced additional expenses of 259 million USD. Finally, the lost rental income is estimated to 1.2 billion USD. In total the impact to economy can be estimated to be between 9.14 billion and 66.2 billion USD. This large variation clearly shows the inherent difficulties in the assessment of economic losses and the associated uncertainties.

B.4 Fatalities

Consequences due to injuries and fatalities are often seen as the most difficult to assess. This is due to the fact that there is no profound basis to estimate the loss of enjoyment, pain and emotional distress. However, a rational basis exists to quantify the economical loss of dependents. Furthermore, the concepts of the human capital and the Life Quality Index together with the Societal Life Saving Costs can be used to assess societal losses due to fatalities. The present section starts with the description of how dependents of World Trade Center fatalities were compensated by the Victim Compensation Fund. Thereafter, the legal practice is considered regarding the compensation of
dependents of fatalities. Thirdly, it is shown how the human capital approach was used by different sources to estimate the loss due to fatalities. After that, the societal losses due to fatalities are assessed by means of the Life Quality Index and the Societal Life Saving Costs. Finally, the present section closes with a review of the fatalities and the $k$-factor, which are associated with the failure of the WTC Twin Towers.

B.4.1 Victim Compensation Fund

After the failure of the WTC towers, the United States of America’s Federal Government enacted the September 11th Victim Compensation Fund (VCF). This fund aims to provide compensation to individuals or relatives of persons, who were killed or physically injured as a result of the events of September 11, 2001. Therefore, the fund distinguishes the compensation of dependents into economic and immaterial losses. As economic loss it considers the lost household income contribution of the fatality. Immaterial losses are reimbursed from the VCF by payments of compensation for pain and suffering.

B.4.1.1 Immaterial loss

Until now, no profound basis is known for the assessment of immaterial losses in order to compensate dependents for pain and suffering due to fatalities. The September 11th Victim Compensation Fund (VCF) decided to award 250’000 USD for each fatality. This corresponds roughly to the compensation, which is received by dependents under the existing federal programs for public safety officers or members of the US military, who were killed while on duty. In addition, 100’000 USD are awarded to spouses and dependent persons [191].

B.4.1.2 Lost dependents’ income

When a person dies, the families and dependents may lose an essential contribution to their income. The expected value of the lost income for a dependent person $i(a)$ can be calculated according to following equation:

$$i(a) = \int_a^{a+a_w(a)} \kappa_T \kappa_U \kappa_{C,S} \delta(a) i_f(a) \, da. \quad (B.1)$$

Here, the lost future income $i_f(a)$ is integrated over the expected remaining working life at age $a$. The future income has to account for growth in earnings. The VCF considers this with the earnings growth rate, as illustrated in Figure B.20. The boundaries of the integral is the age $a$ at death and the expected retirement age $a+a_w(a)$. Depending on the age of the fatality, the VCF determines the remaining working life $a_w(a)$, see Figure B.21. The remaining working life has been determined by using a Markov process based on labor market activity in 1997-1998. Taxes reduce income and its share is depending on the income itself. The after-tax compensable income is determined by applying the average effective combined federal, state and local income tax for the income bracket of the fatality relevant for the state of the domicile of the fatalities. Effective income tax rates, which have been derived from data of the Internal Revenue Service (IRS) for New York, are shown in Figure B.22. In Equation B.1 the reduction by taxes is accounted for by the factor $\kappa_T$. As unemployment is not considered in the calculation of the remaining workforce participation, a 3% unemployment factor is considered by VCF. In Equation B.1 this is considered by $\kappa_U$. $\kappa_{C,S}$ considers the personal expenditures of the fatality according to his or her family status $S$. Because some of the fatality’s income would have been consumed by himself or herself. In Figure B.23, the curves represent the personal consumptions of the fatality as a share of the after-tax income of the fatality (not household income). It is seen that the fraction of personal consumption is highest for childless singles and lowest for married persons with two children. These curves are based on data provided by the Bureau of Labor Statistics [13]. For lower income categories, where total expenditures exceed income, expenditures were scaled to income. Finally, $\delta(a)$ is the discounting function, which capitalizes all future incomes in order to calculate the present value. To ease computational efforts, three mixed after-tax discount rates were used, depending on the age of the fatality. These rates are shown in Table B.22. The discount rates are based on mid- to long-term US treasury securities, adjusted for income taxes, whereby a mid-range effective tax rate has been used. For the computation of the after-tax discount rates, the VCF utilized the income tax rate of New York, which is relatively high compared to other states. This results in a lower after-tax discount rate
and therefore, the present value is higher. The before-tax discount rates, which are shown in Table B.22, are adjusted for inflation and imply real interest rates in excess of inflation of 3.1%, 2.8%, and 2.2%. They depend on the average time to maturity, which is consistent with the average duration of the presumed losses. Finally, the present value of the presumed economic loss is calculated by applying the after-tax discount rate corresponding to the age of the fatality.

Figure B.20 Income growth rate.

Figure B.21 Retirement age and remaining workforce participation.

Figure B.22 Effective income tax rate.
Appendix B – Detailed consequence assessment of the WTC failure

Table B.22  Applied discount rates.

<table>
<thead>
<tr>
<th>Age of Fatality Before-Tax Discount Rate</th>
<th>After-Tax Discount Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 36</td>
<td>5.10%</td>
</tr>
<tr>
<td>36 - 54</td>
<td>4.80%</td>
</tr>
<tr>
<td>&gt; 54</td>
<td>4.20%</td>
</tr>
</tbody>
</table>

Figure B.23  Personal consumption.

According to the prescribed model, the VCF calculated presumed economical losses, which are approximate values to the final compensation. For instance, Table B.23 shows the presumed economical loss of a married fatality with two children. This loss is calculated depending on the age and of the fatality and the income bracket.

The final compensation, which is awarded to dependents, differs from the calculated economic and immaterial loss by the offsets. Offsets are payments, which the dependents have obtained from other sources such as life insurance, pension funds, death benefit programs etc. After subtracting offsets, the VCF awarded an average compensation of over 2.08 million USD per fatality.

Table B.23  Presumed economic and immaterial compensation for a married fatality with two children, from [191].

<table>
<thead>
<tr>
<th>Age</th>
<th>$ 10,000</th>
<th>$ 15,000</th>
<th>$ 25,000</th>
<th>$ 30,000</th>
<th>$ 35,000</th>
<th>$ 40,000</th>
<th>$ 45,000</th>
<th>$ 50,000</th>
<th>$ 60,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>987,184</td>
<td>1,332,626</td>
<td>1,500,699</td>
<td>1,653,341</td>
<td>1,820,980</td>
<td>2,003,481</td>
<td>2,193,816</td>
<td>2,387,691</td>
<td>2,717,316</td>
</tr>
<tr>
<td>30</td>
<td>876,859</td>
<td>1,138,985</td>
<td>1,264,062</td>
<td>1,378,746</td>
<td>1,502,787</td>
<td>1,637,524</td>
<td>1,777,449</td>
<td>1,919,642</td>
<td>2,164,568</td>
</tr>
<tr>
<td>35</td>
<td>809,426</td>
<td>1,012,006</td>
<td>1,109,191</td>
<td>1,198,974</td>
<td>1,294,714</td>
<td>1,398,492</td>
<td>1,505,791</td>
<td>1,614,433</td>
<td>1,804,594</td>
</tr>
<tr>
<td>40</td>
<td>762,203</td>
<td>926,921</td>
<td>1,005,217</td>
<td>1,078,306</td>
<td>1,154,971</td>
<td>1,237,850</td>
<td>1,323,149</td>
<td>1,409,194</td>
<td>1,562,345</td>
</tr>
<tr>
<td>45</td>
<td>715,430</td>
<td>843,187</td>
<td>903,549</td>
<td>960,314</td>
<td>1,019,113</td>
<td>1,082,555</td>
<td>1,147,567</td>
<td>1,212,982</td>
<td>1,330,919</td>
</tr>
<tr>
<td>50</td>
<td>674,689</td>
<td>771,012</td>
<td>816,335</td>
<td>859,097</td>
<td>903,139</td>
<td>956,611</td>
<td>999,116</td>
<td>1,047,834</td>
<td>1,136,315</td>
</tr>
<tr>
<td>55</td>
<td>640,646</td>
<td>710,478</td>
<td>743,227</td>
<td>774,251</td>
<td>805,974</td>
<td>840,132</td>
<td>874,905</td>
<td>909,761</td>
<td>973,617</td>
</tr>
<tr>
<td>60</td>
<td>610,516</td>
<td>657,606</td>
<td>679,565</td>
<td>700,038</td>
<td>721,640</td>
<td>744,544</td>
<td>767,960</td>
<td>791,232</td>
<td>834,050</td>
</tr>
<tr>
<td>65</td>
<td>590,768</td>
<td>622,015</td>
<td>635,711</td>
<td>650,634</td>
<td>664,669</td>
<td>680,196</td>
<td>695,802</td>
<td>711,444</td>
<td>740,100</td>
</tr>
</tbody>
</table>

Although the VCF was initially much criticized, see e.g. [208], it finally was accepted broadly as a just assessment of compensation. More than 98% of the eligible relatives of the fatalities applied for compensation from the VCF and waived any right to file a lawsuit for damages sustained as a result of the events of September 11, 2001, [145] and [123]. Using the frequency distribution of the age of the WTC fatalities and an average gross income of 125’000 USD (see Annex B.3) together with the VCF’s presumed economical and immaterial losses the average compensation payments may be calculated. This yields 2.49 million USD as compensation for a married fatality with 2 dependent...
The average compensation for the dependents of a married fatality with one child is 2.35 million USD and 2.21 million USD for a childless married fatality. Furthermore, the average compensation for the dependents of a single with child will be 1.79 million USD and 1.35 million USD for a childless single. It should be noted that for the evaluation of these figures the same age distribution and average income has been used for all fatalities’ family statuses since further information was not available. However, these characteristics may be correlated with the family status so that these relations should be used in a more thorough analysis.

Table B.24 Illustration of economic and immaterial loss calculation, from [192].

B.4.2 Legal practice

Although the economic loss for dependents of fatalities may be assessed, there is no uniform basis for the assessment of compensation for pain and suffering. Therefore, the amounts of compensation awarded by courts or settlements vary, not least among countries.

In June 3, 1998, a railway accident in Eschede (Germany) caused the death of 101 people and injured 119 persons. In order to reduce bureaucracy and resulting delays and in order to avoid law suits, the Deutsche Bahn (German Railway Incorporated) compensated for economic and immaterial losses. For immaterial losses it offered the dependents 15'345 Euro (approx. 18'000 USD) as compensation for pain and suffering. This is three times the maximum amount, which can be claimed according to current German law. 5'115 Euro can be claimed by dependents, if it can be proven that they were put into an abnormal psychological state. In addition, the Deutsche Bahn compensated for treatment expenses, loss of earnings and material losses. In cases, where a young family lost the father, the Deutsche Bahn pays monthly rents to the dependents. The economical damage was estimated to 150 million DM (76.7 million Euro), whereby two thirds were due to the shortfall from the temporary closedown of the whole ICE-1 train fleet [193].
On July 25, 2000 the Concorde crashed near Paris (France) and 113 persons died. 97 passengers were from Germany and after negotiations the airline company compensated the fatalities’ dependents. Some dependents received compensation far more than 500’000 USD. The details are kept confidential; however, sources report that the total compensation for pain and suffering amounts to 100 million USD [57] and [181]. This sum, which is large for European cases, was achieved by the fact the claimants threatened to file a law suit in the USA.

Generally, in Switzerland, 30’000 CHF (approx. 24’000 USD) is paid as compensation for pain and suffering and 7’500 pounds (approx. 14’000 USD) in Great Britain [180]. According to Willingmann [205], the difference of the compensation for pain and suffering between the European and US American practice origins from the fact that in the USA liability tries to outweigh immaterial compensations, which in Europe is carried by social insurances. Therefore, the compensations paid in the USA for pain and suffering are not comparable with European practices.

In [206], the intention of the creators of the Bürgerliches Gesetzbuch (BGB), the German civil code, with respect to the compensation for immaterial losses is given. Willingmann quotes that “from monetary compensation for immaterial losses, ‘only the bad elements of the society would benefit; lucre, selfishness and greediness would increase, and numerous harassing processes would be exerted from dishonest motives.’” (This quotation was translated from German by the authors).

The families of three American students which were killed in a suicide bombing in Israel received 54 million USD in compensation. After the accidental bombing of a Chinese embassy in Serbia, the United States government paid 1.5 million USD to each family [133].

In 1988, a Pan Am Boeing 747 crashed in Lockerbie after a bomb exploded aboard; 270 persons died. One year later in Niger, a DC-10 passenger jet of the French airline UTA crashed after an explosion. More than ten year later, the Lybian government paid 10 million USD per fatality to the families of the Lockerbie fatalities. 1 million USD were received by each family of the UTA victims [150].

In February 1996, Iran and the United States settled Iran's claim, which was filed before the International Court of Justice. The law suit concerned the shoot down of Iran Air flight 655 on July 3, 1988. The families of each fatality obtained 300’000 USD when the fatality was contributing to the household income. Otherwise the families received 150’000 USD [203].

B.4.3 Human Capital

The human capital is the stock of valuable and useful skills and knowledge accumulated during education and training. It is the capacity of a person to yield financial capital in terms of earning. Two reports evaluate the lost human capital by calculating the lost income. They give close insight in the evaluation procedure. These reports were published by the Comptroller of New York City [118] and the Federal Reserve Bank of New York [70].

B.4.3.1 Comptroller of New York City

In [118] the Comptroller of New York City estimates the lost income. According to [118], the average annual income was 130’000 USD. With a retirement age of 65, the average lost working life is assessed to 25.15 years, see Table B.25. Therefore, the report calculates for 2’672 fatalities, a total of 8.7 billion USD in lost income or 3.27 million USD per fatality. Implicitly, the report assumes that the future growth of income is equal to the price rise and does not account for discounting. Furthermore, the report estimates life insurance payouts to 2.63 billion USD, where it is assumed that 10% of the fatalities had a 1 million life insurance policy, 15% were insured with 500’000 USD and the rest with 250’000 USD.

Table B.25 Average lost working life.

<table>
<thead>
<tr>
<th>Age Group</th>
<th>&lt;31</th>
<th>31 - 40</th>
<th>41 - 50</th>
<th>51 - 60</th>
<th>&gt; 60</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage (out of 2,819)</td>
<td>19.5%</td>
<td>36.8%</td>
<td>27.7%</td>
<td>13.0%</td>
<td>3.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>Average Age</td>
<td>26.6</td>
<td>35.6</td>
<td>44.8</td>
<td>54.7</td>
<td>65.9</td>
<td>39.9</td>
</tr>
<tr>
<td>Years to Retirement</td>
<td>38.4</td>
<td>29.4</td>
<td>20.2</td>
<td>10.3</td>
<td>0.0</td>
<td>25.2</td>
</tr>
</tbody>
</table>
B.4.3.2 Federal Reserve Bank

Based on an average annual income of 127,000 USD, the Federal Reserve Bank estimates the lost income to 2.8 million per fatality. With an average age of 40 years, the remaining working life is assessed to 22 years. Finally, the lost income is estimated to 7.8 billion USD. For the calculation of the lost income, the report assumes that the increase of income is equal to the increase of inflation; however, no discounting is assumed.

B.4.4 Life Quality Index and Societal Life Saving Costs

An approach to assess consequences due to fatalities has been suggested by Lind [95] and was then extended by Nathwani, Lind and Pandey [109]. There, a compound social indicator, the Life Quality Index (LQI) is introduced. This index is composed of three social indicators, namely the gross domestic product per capita , the life expectancy and the fraction of life spent to earn a living . The LQI is highly correlated with the Human Development Index, an index which is used by the United Nation Development Program to assess a country’s level of development. From the LQI, Skjong and Ronold [174] derive the Implied Costs to Avert a Fatality (ICAF), which has been introduced to optimal design of civil engineering structures by Rackwitz [162]. Furthermore, Rackwitz [163] averages the life saving costs LSC or ICAF over the population’s age distribution to obtain the Societal Life Saving Costs (SLSC).

B.4.4.1 Life Quality Index

Lind [95] introduces a compound social indicator , which is a function of several other social indicators.

\[ L = L(a,b,...,e,...) \]  

(B.2)

When \( L \) is differentiable its total differential is given by following equation.

\[ dL = \frac{\partial L}{\partial a} da + \frac{\partial L}{\partial b} db + ... + \frac{\partial L}{\partial e} de + ... \]  

(B.3)

When \( L \) is only a function of \( e \) and \( g \), then \( dL \) vanishes, if

\[ dL = 0 \Rightarrow \frac{dg}{de} = -\frac{\partial L}{\partial e}. \]  

(B.4)

This implies that a change in \( e \) should be compensated by a change in \( g \). Assume that \( L \) is the product of a function of \( g \) i.e. \( f(g) \), which quantifies life quality, and a function \( h(t) \) of the time to enjoy life \( t = (1-w)e \). The latter considers the length of life. In this equation, \( w \) is the proportion, which is spent in paid work. An individual can increase leisure time either by increasing the life expectancy \( e \) or by decreasing the time spent at work, which generally will reduce its income. The concept of LQI assumes that each individual chooses \( w \), so that \( L \) is maximal. Then \( L \) can be formulated as:

\[ L = f(g)h(t) \]  

(B.5)

\[ \frac{dL}{L} = \frac{g}{f(g)} \frac{df(g)}{dg} \frac{dg}{g} + \frac{t}{h(t)} \frac{dh(t)}{dt} \frac{dt}{t} = k_g \frac{dg}{g} + k_t \frac{dt}{t} \]  

(B.6)

Assuming \( k_g / k_t = const. \) and that \( g \) is proportional to the time spent at work, Nathwani, Lind and Pandey [109] derived the Life Quality Index \( L \). The factor \( (1-w)^{1-w} \) can be omitted as a good approximation.

\[ L = g^w e^{1-w}(1-w)^{1-w} \approx g^w e^{1-w} \]  

(B.7)
Taking the \( \frac{1}{1-w} \)-th root and dividing by \( q = w/(1-w) \), removes a minor inconsistency of the original form because persons with the same \( g \) and \( e \) but larger \( w \) would have a higher LQI.

\[
L = \frac{g^q}{q^e} \quad \text{with} \quad q = \frac{w}{1-w}
\]

From the LQI, a general acceptance criteria can be derived, which requires that the relative change in life quality from any action aimed to safe lives should be at least zero.

\[
\frac{dL}{L} \geq 0
\]

Applying this criteria to the established relation of \( g \), \( e \) and \( w \), the following is achieved:

\[
\frac{dg}{g} + \frac{1}{q} \frac{de}{e} \geq 0.
\]

### B.4.4.2 Implied Costs of Averting a Fatality

Based on Equation B.10, Skjong and Ronold [174] derived the upper limit of the reduction in GDP per capita. This is referred as the acceptable cost per life year saved.

\[
-g \frac{de}{q e} \leq \frac{\Delta e}{q e} \] \quad \text{(B.11)}

Multiplying the acceptable cost per life year saved with the number of saved life years, one obtains the Implied Cost of Averting a Fatality (ICAF).

\[
ICAF = \left| \Delta g \right|_{\text{max}} \Delta e = \frac{g}{q} \frac{\Delta e^2}{e} \] \quad \text{(B.12)}

When it is assumed that the average saved life years of an arbitrary individual equals half the life expectancy at birth \( \Delta e = e/2 \), than the equation for ICAF can be simplified.

\[
ICAF = \frac{g e}{4 q} \] \quad \text{(B.13)}

In 2001, the US gross domestic product per capita was 35,277 USD and life expectancy at birth was 76.9 years [189]. With \( w = 0.18 \), see [163], one obtains 3.1 million USD per fatality.

### B.4.4.3 Societal Life Saving Costs

In Rackwitz [163], a modified ICAF is introduced. This ICAF or Societal Life Saving Costs (SLSC), is obtained by separating the variables of Equation B.10. Then the inequality is integrated from \( g \) to \( g + \Delta g \) on one side and from \( e \) to \( e + \Delta e \) on the other side. These operations yield the yearly costs \( \Delta C = -\Delta g \) to extend a person’s life by \( \Delta e \) year.

\[
\Delta C = -\Delta g = g \left[ -1 - \left( 1 + \frac{\Delta e}{e} \right)^{\frac{1}{q}} \right] \] \quad \text{(B.14)}

Since \( \Delta C \) are yearly costs they have to be multiplied by \( \Delta e \) to obtain the total life saving costs LSC.

\[
LSC(\Delta e) = g \Delta e \left[ -1 - \left( 1 + \frac{\Delta e}{e} \right)^{\frac{1}{q}} \right] \] \quad \text{(B.15)}
Averaging the life saving costs over the age distribution, one obtains the Societal Life Saving Costs (SLSC).

\[
SLSC = \int_0^\infty LSC(e(a)) f_a(a) \, da
\]

(B.16)

In this equation, \( e(a) \) is the conditional remaining life expectancy given a person is \( a \) years old. \( f_a(a) \) is the population’s age probability density function. A good approximation of the SLSC is the life saving costs for a person at age equal to the half life expectancy.

\[
SLSC \approx LSC(e/2)
\]

(B.17)

In [163], the value of SLSC for USA is indicated as 870’000 USD. When cohort life tables are used, the life expectancy is higher and SLSC becomes close to 1 million USD, see [163].

### B.4.5 Consequences due to fatalities

"2'749 Unless something unexpected happens, this figure could well be the final count that history books will record as the number of victims who died in the World Trade Center attack, …" [146]. Shortly after September 11, the official number of missing persons was as high as 6'700. On October 23, 2002 this number was reduced to 4’764 and in the aftermath the number dropped to the level of which the New York Times estimated already in October 2001. The number 2’749 comprises the persons, who were present at the WTC in addition to 92 persons on board of American Airline flight 11, the plane which hit into WTC1. 65 persons were on board of United Airlines flight 175, the plane that hit WTC2.

Furthermore, on September 11, 2001, 125 persons died at the Pentagon and 59 persons were on board of American Airline flight 77, the plane which hit into the Pentagon. Finally, 40 persons died on board of United Airline flight number 93, the plane which crashed in Pennsylvania [173]. The internet site [173] is also referred by the White House Commission on the National Moment of Remembrance and the IRS.

![Figure B.24 Frequency distribution of the age of the fatalities by employers.](image-url)
Even, when the number of 2'749 fatalities will not be changed in the future and will become the official number of fatalities, which history books will address to September 11, this number will still be an estimate. The real number of people, which died in New York on September 11, 2001, will probably never be known exactly.

While rescuing others, 343 fire men lost their lives. During the clean-up, no fatality or career-ending injury was reported, luckily, despite the fact that numerous people with heavy construction equipment worked round-the-clock [121]. [74] reports that 8 men died during the construction of the WTC, whereby none of them were iron workers, whereas in [60] it is indicated that 60 people were killed during construction. Therefore, the reconstruction of the WTC may involve further fatalities although that number seems to be small, when compared to 2'749. Currently, the effects of harmful substances in the air at ground zero such as asbestos, benzene, dioxin and polychlorinated biphenyl (PCB) on the health of persons cannot be reliably predicted and thus bear further risks.

Figure B.25  Lost employees per employer.

Figure B.15 in Annex B.3.1.5 illustrates the age distribution of the fatalities and compares this with the age distribution of New York City’s population. The mean value of the WTC fatalities’ age is 39.7 years and the standard deviation is 10.2 years. Figure B.24 distinguishes the age frequency distribution by employers who experienced the largest losses. Cantor Fitzgerald suffered a loss of 658 employees, the New York Fire Department lost 343 of its members, Marsh & McLennan lost 295 persons of its staff and Aon experienced a loss of 176 employees [173]. It is seen that none of the four employers lost staff personal younger than 20 or older than 75. Besides this, no general observations can be found. It seems that the age distribution of Cantor Fitzgerald has a single mode at 34 years, the age distribution of the fire and police men has two and the age distribution of Marsh and McLennan has even three modes. Furthermore, Aon employees are older, when compared with the total population of fatalities and Cantor Fitzgerald employees are somewhat younger. Generally, the employees’ age distribution of an individual enterprise will strongly depend on its economic situation and the development of the economical sector.

Figure B.25 shows the number of enterprises which suffered a certain loss of employees. 110 enterprises lost only one employee. This is more than 55.8% of all enterprises. However, these 110 fatalities constitute only 4.5% of the total number of fatalities. Furthermore, it is seen that around 90% of the affected enterprises lost less than 10 co-workers, which correspond to 13% of the total
Fatalities. The four enterprises with the highest losses represent almost 60% of the fatalities although they constitute 2% of the affected enterprises. Cantor Fitzgerald’s fatalities alone accounts for 27% of all fatalities. Finally, 77% of the fatalities were male and 23% female.

B.4.6 k–factor

The $k$-factor is defined as the probability that a person dies given an incident such as the structural failure of a high-rise building. The $k$-factor is assessed by the number of fatalities $N_f$ divided by the total number of affected persons. For high-rise buildings this is the number of people present in the building $N_p$.

$$k = \frac{N_f}{N_p} \quad \text{(B.18)}$$

In the case of office buildings it is meaningful to refer the number of present people to the gross floor area and the time of day. The number of employees working in a building may be derived by the building’s gross floor area $A$ and the floor space, which is provided per employee $A_E$, see also Annex B.3.1.5. When the area $A_E$ does only account for fully occupied buildings, the available space has to be considered by the occupancy rate $O$.

$$N_p = \frac{A}{A_E} O \quad \text{(B.19)}$$

People working in an office building are not present at all times. Variations depend on the day, time of day and the building’s usage. For instance, in a general office building the proportion of people, working on a weekday late at night is rather low, while the number of present people is high in hotels or residential buildings. With data from the Wall Street Journal [207], the percentage of people present is estimated to 50.8%. It should be noted that September 11, 2001 was the mayoral primary election day in New York City and some of the employees went to vote for the mayor succeeding Rudolph Giuliani before going to work [129].

The number of present people $N_p$ has also to include the staff for the operation and maintenance of the buildings and where necessary visitors, as well.

The analysis of the data from the Wall Street Journal, concerning the failure of the WTC Twin Towers, results in a mean value of the $k$-factor equal to 22%. By differentiating this factor for each of the towers, 16.7% is obtained for WTC1 and 19.5% for WTC2. This difference is not surprising because the towers were hit by the planes at different floor levels and at different times. Evaluating the $k$-factor for the area below the impact area one obtains 9.1%. This is so because an enterprise situated below the impact area lost both employees. If these two persons are neglected in the calculations, then the $k$-factor becomes zero. This is what was generally observed, see also [58]. Considering the data for floors above the impact area, one obtains a single data set which indicates a factor $k$ of 100%. Although only few data is available, one can assume that $k$ is 100% for floors above the impact area because, if the proportion of present people is evaluated with the assumption that $k = 100\%$, then almost the same value is obtained as for the whole North Tower without this assumption. Assuming that the persons are uniformly distributed over all floors leads to a simple model for $k$:

$$k = f_{max} - f_e - 1 \quad \text{(B.20)}$$

In this equation, $f_{max}$ is the total number of floors in the building and $f_e$ is the floor at which the failure event takes place and blocks the egress system.

Evaluating this expression for WTC1, one obtains 17.2%, which is close to the value derived from the data (16.7%). Applying this model for WTC2, 30% is obtained, which is very different to 19.5%, the value derived from the data. However, WTC2 was the second tower that has got hit. Persons in that building had the possibility to escape using the elevators; however, not everybody did. As already mentioned, Equation B.20 is a simple approach to evaluate the $k$-factor. For cases where the
underlying assumptions for Equation B.20 do not apply, the more general Equation B.18 has to be evaluated.

**B.4.7 Summary**

For the assessment of the consequences due to fatalities, first of all the number of fatalities has to be assessed, which can be modeled by the number of present people in a building and the $k$-factor. The latter is the probability that a person dies in case of an incident. The number of people present in an office building can be evaluated as a function of the office space provided by the building and a reference value for the space, which is provided per employee. Furthermore, the model has to consider the variations with respect to time of day, day and use.

For the USA, the SLSC is 1 million USD, if the life expectancy is evaluated according to cohort life tables. This amount should be considered for each fatality within a risk based decision framework. However, the amount which should be considered may also depend on the societal and legal system of the considered country. In case of the failure of the WTC Twin Towers the Victim Compensation Fund compensated the dependents of the fatalities. Based on the compensation scheme an average compensation of 2.0 million USD per fatality can be calculated summarizing to a total cost of compensation of 5.5 billion USD. The difference between the compensation of the VCF and the SLSC stems from the fact that the SLSC averages over all individuals of the society, which considers children, unemployed persons, retired persons and other nonworking persons. Furthermore, the average American citizen earns less than a World Trade Center employee and lastly the SLSC does not consider immaterial losses, which e.g. were paid by the VCF.

**B.5 Environmental consequences and loss of cultural assets**

**B.5.1 Environmental impact**

At ground zero the air quality was bad for weeks due to the ongoing smoldering fires. The resulting plume was so hot from the fire beneath that it rose directly into the air where it dissipated. Therefore, it is said that the plume had only localized effects and the air quality across most of Lower Manhattan quickly returned to normal conditions. In order to follow up the health effects, a health registry was set up in September 2003. It is intended to cover residents and employees of Lower Manhattan, rescue and clean-up workers and anyone who simply was in Lower Manhattan on September 11, 2001. Until March 2004, the 20 million USD registry had more than 25'000 persons registered and aims to follow the health condition of as many as 200'000 persons [143] and [119]. A future compensation of people affected by the air quality bears future risks, underlined by the fact that more than one thousand firemen have filed lawsuits against the City of New York asking for 12 billion USD [138]. Information regarding the environmental consequences due to the WTC failure can also be found in [76].

Apart from the environmental impact to the health of people, no report was found, which accounted for the environmental consequences with regard to animals, plants and water.

**B.5.2 Loss of cultural assets**

**B.5.2.1 Art objects**

Among the lost and destroyed goods at the WTC site are famous works of art. They were bought by the Port Authority in order to furnish the WTC complex appropriately and by the tenants to furnish their office rooms. There was the WTC Tapestry of Joan Miró, the Bent Propeller of Alexander Calder and Fritz König’s Grosse Kugelkaryatide, Roy Lichtenstein’s Entablature and works of Louise Nevelson, Pablo Picasso and Auguste Rodin. From the latter artist there were approximately 300 masterpieces in the rooms of Cantor Fitzgerald, which had a collection. Among these were the Thinker, a 28 inch (71 cm) bronze cast of which several dozen casts are held by museums and private collectors. During the clean-up, the statue was recovered by a fireman. However, it disappeared and probably was stolen after that. The insured value of the art works is finally estimated to around 100 million USD [149], [94] and [2].

**B.5.2.2 St. Nicholas**

St. Nicholas was a Greek Orthodox Church situated just 250 ft (76 m) south of the WTC complex at 155 Cedar Street. It was 22 feet (6.71 m) wide in the front and 20 feet 11 inches (6.38 m) in the back.
It was about 56 feet (17.1 m) long, 35 feet (10.7 m) tall and on three sides it was bounded by a parking lot. Around year 1832, the tiny church was constructed as a residential building and was later converted into a tavern and then in 1916 into a church. On the building’s top floor the parish kept the relics of the three saints, namely St. Nicholas, St. Katherine and St. Sava. Next to these cultural assets the church housed icons, which were given by Czar Nicholas II of Russia [130] and [75].

**B.5.3 Summary**

20 million USD are being spent to establish a health registry, aiming to follow the development of the health of up to 200’000 persons. However, the 20 million USD will not cover future compensation payments to these people. For instance, more than one thousand firemen have filed lawsuits against the City of New York asking for 12 billion USD.

Moreover, the towers’ failures destroyed artworks and cultural assets. The artworks situated within the WTC were insured to an estimated 100 million USD. However, this does not account for the non-reproducible loss of masterpieces of Miró, Picasso, as well as the cultural assets, which were stored in the tiny Greek Orthodox church St. Nicholas.

**B.6 Risk perception**

The failure of the World Trade Center towers also led to changes in risk perception and not only in the American society. Decision makers affected by such changes may assess risks differently in the future.

However, in Chapter 3 it is underlined that when decisions are made on behalf of society, the risk perception of individuals is not relevant.

**B.7 Total effect to society and liability**

**B.7.1 Total effect to society**

With regard to the design of civil engineering structures or the assessment of risk associated to such facilities, it is often argued that consequences to the society have to be considered. Based on a macroeconomic approach, consequences to the society do not only reflect costs and losses, which are associated to the adverse event. When such an approach is pursued, then also positive effects to the society have to be considered. For instance, when a natural hazard, e.g. an avalanche or tornado, destroys buildings, these buildings are often reconstructed. This will lead to a higher demand in the construction sector and enterprises will have higher profits and might need more employees. Furthermore, local, state or federal administration will get higher tax incomes from employees and enterprises of the construction sector.

The preceding chapters are focused on the assessment of damages and losses experienced by individuals, enterprises and organizations of the society. However, some individuals, enterprises and organizations might be affected in a positive way by an incident.

When an incident occurs, then some individuals, enterprises and organizations may increase their economic activity to remove and clean-up the damage and restore the original state. Therefore, enterprises may increase their profits and additional persons might be employed. Furthermore, the governmental administrations collect more taxes. The total effect on the society due to an adverse event is the total loss and damage to all individuals, enterprises and organizations. But it considers all positive effects, as well. In the extreme, the net effect may be zero or even be positive. In such cases the positive effects are equal to or larger than the costs. The following example represents such a case.

In February 1993, a bomb attack was carried out in the basement of the World Trade Center complex. Following an assessment of the incident, the Port Authority estimated that the incident generated jobs and economic activity, which would result in a net gain of 200 million USD, see [36].

Another source of positive effects results from the fact that reconstructed facilities generally show improvements, when compared to the structure it replaces. This may allow a higher productivity due to technological improvements or by a better design concept. In the case of the WTC failure, the City
Appendix B – Detailed consequence assessment of the WTC failure

of New York takes advantage to redesign and improve the transportation infrastructure in Lower Manhattan. Within a risk based decision framework for the optimum design of structures, such improvements are rather difficult to predict because structures are generally built according to the state-of-the-art. The technological progress and boundary conditions such as the economic development are rather difficult to predict, especially when a time frame of decades is considered.

Considering the preceding discussion, it follows that engineering decision making should not try to optimize the total effect on the society due to adverse events because, the total effect is small, sometimes zero or may even be positive. In the case, where the net effect is zero, the objective function is indifferent to the structural design and any design is a valuable one. When the net effect is even a positive gain, then the least reliable structure would be the optimal one. This conclusion contradicts engineering experience. Therefore, the total effect on the society can not be used in order to derive acceptance criteria. Hence, a different objective function is required for the risk based approach.

B.7.2 Liability

If an incident occurs and even if the total effect to the society is zero, then a part of the society is affected negatively. In that case the negatively affected persons’ right to be safeguarded was not followed. The right to be safeguarded follows directly from the ethical principle of restricted liberty. Restricted liberty says that everyone is free to do what he or she pleases to do unless no other person is harmed. If this principle is violated than the principle of strict liability forces actors unconditionally to repair or compensate persons which were harmed by their actions. The implication of these principles on the legal systems and decision making are further discussed in [212]. Without liability the decision maker may take into account the consequences directly related to his or her utility and neglecting the follow-up consequences imposed to others. Considering strict liability the decision must take into account the negative effects of his or her action upon others.

Comparing the approaches considering the total societal effect with the approach considering the liability of the decision makers’ actions, it is seen that in order to derive acceptance criteria, the owners and operators of civil engineering structures should consider within the decision problem formulation the follow-up costs of other individuals, enterprises and organizations of the society which may be affected by the construction, operation and decommissioning of the facility. In such cases the owner and operator of such facilities take over the responsibility of their activity with respect to the society.
C Basics in macroeconomics

The preceding chapters assessed the socio-economic consequences associated to the failure of extraordinary structures. In order to assess these consequences, concepts known from macroeconomics were used without having them introduced before. The present chapter gives a short introduction to these utilized concepts, see also [81] and [170].

To start with the gross domestic product (GDP) and the value added is introduced and its use for the modeling of economic consequence is discussed. Then the multiplier effect is explained followed by the price indexes such as the building cost index (BCI).

C.1 GDP

The state of a society may be assessed using social indicators such as population growth, workforce, unemployment rate, inflation, etc. However, the most important indicator is the gross domestic product (GDP). The nominal GDP considers actual market prices, whereas the real GDP refers its prices to a base year in order to eliminate the effect of inflation.

Generally, the GDP is used as an indicator for the prosperity or performance of a country neglecting measures such as life expectancy, political system, literacy, human rights, education, recreation time, safety etc. In order to overcome this, compound indicators may be constructed considering more than one indicator. One compound indicator is the Life Quality Index (LQI), which permits to derive acceptable safety levels, see B.4.4.

There are two approaches in order to determine the GDP. The flow-of-product approach sums up all final products, which are bought and used by consumers. The market value of these goods and services correspond to the GDP. The second approach to determine the GDP is the so called earning or cost approach. This approach analyses the costs for the production of final products, e.g. wages, rents, interest and profits. Both approaches are equivalent and lead to the same GDP.

For the purpose of illustration, Figure C.1 shows an economy circuit of a simple economy. This economy consist of households, which offer productive factors at the factor market, such as labor, land, capital, etc. These factors are sold to enterprises, which buy the factors in order to produce goods and services. Then the products are sold to the households. It is seen that with the upper loop, the GDP is measured with the flow-of-product approach, whereas the lower loop permits to determine the GDP using the earning or cost approach.

As mentioned before, Figure C.1 shows an oversimplified economy. Generally, if the flow-of-product approach is utilized, then the GDP can be further divided into personal consumption \( C \), gross privat domestic investment \( I \), government consumption and investment purchase \( G \) and net export \( X \).

\[ GDP = C + I + G + X \]  

(C.1)

When the earning or cost approach is used then the GDP is determined by 1.) wages, salaries and supplements, 2.) net interests, 3) rental income, 4) indirect business taxes, 5) depreciation, 6) income of unincorporated enterprises and 7) corporate profits enterprises.

In addition to the GDP there is the net domestic product (NDP). The NDP is obtained by subtracting depreciation from GDP. Generally, the NDP is a better measure of an economy’s output because it
considers the net investment. In contrast to the NDP, the GDP considers only the gross investment, e.g. new buildings, new machines; however, it neglects depreciation, e.g. obsolete machines put out of service. GDP is more often used than NDP because the depreciation is difficult to estimate. From experience, the NDP amounts to around 90% of the GDP.

The gross national product (GNP) is the total output produced by labor and capital owned by a society or nation. Whereas, the GDP is the output produced with labor and capital, which is located within the border of a country.

### C.1.1 Value added

If the GDP is assessed then the problem of double counting may arise. This problem is best illustrated by means of a simple example. Assume a simple economy, which only consists of two enterprises. The first enterprise constructs buildings worth 1 million USD every year. Therefore, it utilizes material worth 0.5 million USD, which are produced by the second enterprise. The GDP is defined as the total production of final goods and services. In the present example this is 1 million USD (the market value of the buildings). It excludes intermediate goods. In the present example this is represented by the material produced by the second enterprise. When the cost approach is used one may be inclined to add the costs of both enterprises together and obtain 1.5 million USD. However, national accounting considers only the firm’s value added. This is the difference in the firm’s sales and its purchases of goods and services obtained from other firms. Utilizing this concept, we obtain a GDP of 0.5 + 0.5 = 1 million USD and the value added of the enterprises producing intermediate goods is not double counted.

### C.1.2 Shortcomings of the GDP

The gross domestic product is the soundest indicator for the total output of an economy; however, there are some shortcomings. For instance, it does not consider activities at home, which produce goods and services, such as housekeeping and neighborly help, neither of them is reported in payrolls. The GDP also does not reflect illegal employment, and illegal activities e.g. drug dealing, smuggling, etc. However, these shortcomings are not relevant for the assessment of consequences due to the failure of civil engineering structures.

### C.2 Multiplier effect

The multiplier model is the simplest model but the most influential in order to model the effect of extrinsic influences upon an economy. Despite its simplicity, this model is able to describe the economy’s behavior, when it is subjected to an economic impact and the essence remains valid, even when the model is extended. The simplified economy model is based on the following assumptions.

Firstly, it is assumed that labor force is always available. This means that there are always unemployed persons. The second and most crucial assumption is that impacts from financial markets and monetary policy onto the economy are neglected. This does also imply that interest rates remain unchanged. Thirdly, trade with other countries is omitted. Moreover, the aggregate supply side is not
Multiplier effect

considered and finally, investment is treated as an exogenous force, which is modeled to be independent from the GDP.

Considering these assumptions, the total spending can be formulated as a function of the output GDP. The total spending $S$ consists of the investment $I$, which is assumed to be independent of the GDP and the consumption. In the multiplier model, the consumption is modeled as a linear function of the GDP, see Equation C.2.

$$S = I + C(GDP) = I + C_0 + MPC \cdot GDP$$

(C.2)

The economy is in equilibrium when the total spending is equal to the total output $S = GDP$. Substituting $S = GDP$ into equation C.2, one obtains:

$$GPD_1 = \frac{1}{1 - MPC}(I + C_0)$$

(C.3)

In Figure C.2a this equilibrium is indicated by the point $E_1$. If an increase of investment of $\Delta I$ is considered, a new equilibrium is obtained indicated by $E_2$.

$$GPD_2 = \frac{1}{1 - MPC}(I + \Delta I + C_0)$$

(C.4)

Taking the difference of Equation C.3 and C.4 one obtains:

$$\Delta GDP = \frac{1}{1 - MPC} \Delta I.$$  (C.5)

It is seen that if the investment is increased by $\Delta I$, then the GDP is increased by $1/(1-MPC)$ times $\Delta I$. $1/(1-MPC)$ is the multiplier which gives the economic model its name. In these equations, MPC is the marginal propensity to consume. It is the extra amount people consume, when they receive an extra dollar of disposable income, or in this model an extra dollar of GDP. As MPC is smaller than one, the multiplier is larger than one and if for instance $MPC = 2/3$, then the multiplier becomes 3.

Figure C.2a illustrates the multiplier model and Figure C.2b illustrates the multiplier model by means of aggregate demand and aggregate supply curves.

Another illustrative approach to the multiplier model considers an unemployed person, e.g. a mason who is hired to construct a building worth 100'000 USD. The mason and the brick stone fabricant together obtain 100'000 USD. When they have a marginal propensity to consume of 2/3, they consume 66'666 USD in goods and services. Persons offering these goods and services consume from the amount they obtained, as well 2/3. This process continues infinitely often. Finally, the total output may be written as

$$\Delta GDP = \Delta I + MPC \cdot \Delta I + MPC^2 \cdot \Delta I + ... = \Delta I \sum_{n=0}^{\infty} MPC^n = \Delta I \frac{1}{1 - MPC}.$$  (C.6)

In Equation C.6, use has been made of the geometric series, see Equation C.7.

$$\sum_{n=0}^{\infty} r^n = \frac{1}{1 - r}.$$  (C.7)

Generally, by means of improved models for aggregate supply and aggregate demand curves, more accurate multipliers may be derived at the price of more complex models, loss of insight and more difficult calculations.
Appendix C – Basics in macroeconomics

C.3 Price index and GDP deflator

C.3.1 GDP Deflator

When the gross domestic product of different years is compared, the real GDP is used. The real GDP is obtained from the nominal GDP and the GDP deflator. The nominal GDP is the total money value of goods and services produced in a given year, and the real GDP is the nominal GDP valued in prices of a base year. Hence, the GDP deflator is a price index.

\[
GDP_{\text{real}} = \frac{GDP_{\text{nominal}}}{GDP_{\text{deflator}}}
\]  

(C.8)

A price index is a macroeconomic indicator describing the change of prices with time. It is based on a market basket, which contains goods and services. In order to construct a price index, the prices of goods and services have to be weighted by the commodity’s economic importance. For the case of the consumer price index, the weights are proportional to the total spending of the considered item.

C.3.2 Inflation and Consumer Price Index CPI

The rate of inflation is defined as the rate of change in price level and is given by following equation.

\[
\frac{r_{t}}{t} = \frac{p(t) - p(t-1)}{p(t-1)}
\]  

(C.9)

Where \( r_{t} \) is the rate of inflation and \( p(t) \) the price level in year \( t \).

The most widely used measure for inflation is the consumer price index (CPI). It measures the costs of a market basket of consumer goods and services. The basket contains e.g. food, clothing, housing, transportation, energy sources etc. Besides the CPI, the production price index PPI is commonly used to measure inflation, and the GDP deflator is sometimes used to supplement CPI and PPI. This is supported by data for Switzerland, which show a coefficient of correlation of 85% between the annual change in GDP deflator and annual change in CPI. Hyperinflation is known as a monthly
price rise of 50% or more, whereas deflation is the opposite of inflation. This is when the general level of prices is falling.

C.3.3 Numerical example
Consider a market basket consisting of three goods, namely steel, concrete and timber. A market analysis shows that from the material costs spent for the construction of buildings, 60% is assigned to concrete, 30% to steel and 10% to timber. Using the base year of 1999 and setting the price for each good to 100, the price index for the base year is obtained to be 100, as well. When in the following year the price for concrete rises 10%, steel rises 5% and the price for timber falls 10%, then the price index for 2000 is obtained to $(0.6 \times 1.10) + (0.3 \times 1.05) + (0.1 \times 0.90) = 1.065$. Hence, the overall price for construction materials rises in average 6.5% from year 1999 to 2000.

C.3.4 CPI for USA
The Bureau of Labor Statistics publishes two consumer price indexes, namely the CPI for urban wage earners and clerical workers, which prescribes the inflation perceived by 32% of the US population and secondly the CPI for all urban consumers and the chained CPI for all urban consumers (C-CPI-U), see [13]. The latter covers around 87% of the US population consisting of wage earners and clerical worker households, groups such as professional, managerial, and technical workers, the self-employed, short-term workers, the unemployed, and retirees and others not in the labor force. The CPI’s market basket is based on prices for food, clothing, shelter, and fuels, transportation fares, charges for doctors’ and dentists’ services, drugs, and other goods and services that people buy for day-to-day living. The prices are collected in 87 urban areas from about 50’000 housing units and approximately 23’000 retail establishments, department stores, supermarkets, hospitals, filling stations, and other types of stores and service establishments. Within the price index, all direct taxes associated with the items are included.

Figure C.3 shows the US city average CPI for all urban consumers and its annual change. The index base year is 1967; the 1965’s value was 94.5 and in 2001 the index was at 530.4. This implies a price rise with a factor of 5.61 or an equivalent constant inflation rate of 4.91%.

![Figure C.3](image)

The Bureau of Labor Statistics indicates that the CPI for all urban consumers is based on Laspeyres definition for price indexes.
Appendix C – Basics in macroeconomics

C.3.5 Laspeyres price index
The precedent numerical example in Section C.3.3 illustrates how price indexes can be easily constructed and calculated. Implicitly, it was assumed that the weights are fixed and do not vary with time. Such price indexes are called price index with fixed weights or Laspeyres price index. Generally, they can be formulated as follows.

\[
P_L(t) = \frac{\sum_{i=1}^{n} \alpha_i(0) p_i(t)}{\sum_{i=1}^{n} \alpha_i(0) p_i(0)}
\]

(C.10)

In this equation, \( P_L(t) \) is the price index at time \( t \), \( p_i(t) \) is the price level at time \( t \) of the \( i \)th commodity in the basket. This price level is weighted with \( \alpha_i(0) \), the share of the baskets \( i \)th commodity, which is determined by the base year.

As the weights are fixed with reference to the base year, the Laspeyres price index is not able to consider changes in the preferences of goods and services. This however, may be accounted by a price index formulated according to Paasche.

C.3.6 Paasche price index
The Paasche index is defined as the basket’s value in year \( t \) divided by the base year’s basket value. The difference to the Laspeyres index is that in Equation C.11 the economic importance of each good and service may vary with time and that changes in consumer preferences may be considered.

\[
P_P(t) = \frac{\sum_{i=1}^{n} \alpha_i(t) p_i(t)}{\sum_{i=1}^{n} \alpha_i(0) p_i(0)}
\]

(C.11)

However, both indexes the Laspeyres and the Paasche index may not consider obsolescence of products and their substitution by other products, which are not part of base year’s basket, e.g. consider long playing records and compact disc and the occurrence of pre- and post-tensioned concrete. These indexes may also not account for quality improvements of products such as higher thermal isolation and noise insulation of buildings.

C.3.7 Building and construction price index BCI and CCI
The Engineering News Records [42] publishes both a Building Cost Index (BCI) and a Construction Cost Index (CCI) For the United States of America. These indexes represent a 20-city national average. Both indexes consider materials and labor components, see [42]. The difference between the indexes is due to a different representation of the labor component. The CCI considers 200 hours of common labor and BCI uses 68.38 hours of skilled labor each multiplied by the 20-city wage and fringe average. Both indexes use 25 cwt (1.13 t) of fabricated standard structural steel at the 20-city average price, 1.128 tons (1.02 t) of bulk Portland cement priced locally and 1,088 board-ft (2.56 m³) of 2x4 lumber priced locally. These indexes may be applied to general construction costs, whereby the CCI should be used where labor costs represent an important fraction. The BCI is more widely applicable for building structures.

Figure C.4 shows the building and construction cost index for the United States of America, with base year 1913. Until 1947, the annual change of the indexes fluctuates significantly with extremes of -22% in 1932 and +39.8% in 1916. After 1945, knowledge about macroeconomics influenced decision making of politicians and removed strong fluctuations in economy in general and in the construction sector, as well. After 1947, the annual changes are always positive. From their composition, the BCI and the CCI are correlated, which becomes obvious when the annual change of the indexes are compared. However, the annual change of CCI is in average larger, which implies that costs for labor increased more than for materials. In 1965, the BCI was 627 and in 2001 3'574. This implies a price rise of a factor 5.70 or an average annual change of 4.95%. For CCI, an equivalent constant inflation rate of 5.35% is obtained.
Summary

The Gross Domestic Product (GDP) considers the income of employed persons. Furthermore, it accounts for corporate profits, taxes, rents and capital costs. Therefore, it is the best indicator available to measure the society’s economic loss due to an adverse event. The calculated lost GDP,
represents the economic loss of the employees, employers, landlords, investors and public administrations, without indicating how the loss is distributed among them. In order to avoid double counting, the lost added value should be considered.

The gross domestic product does not consider activities at home, which produce goods and services such as housekeeping and neighborly help because they are not reported in payrolls. The GDP also does not reflect illegal employment, and illegal activities, e.g. drug dealing, smuggling, etc. It seems plausible that the latter need not to be considered in an evaluation of economic losses, whereas the other activities seem to be of minor importance, when economic losses associated to several thousand employees are considered.

Although the basic assumptions for the Multiplier model seem to be very restrictive, it is able to predict the economic behavior when subject to economic impulses. Even when the model is extended, the main essence remains valid. The simplified economy model is based on assumptions, whereas for the case of economic losses due to structural failure some of them seem to be fulfilled. The first assumption is that labor force is always available, which is supported by unemployment records, and when needed, the labor force can be acquired from abroad. Secondly, it is assumed that impacts from financial markets and monetary policy onto the economy are neglected. This implies that interest rates remain unchanged. Thirdly, trade with other countries is omitted. Fourthly, the aggregate supply side is not considered. These assumptions (2–4) seem to be fulfilled, when the economic impact due to an incident is small, when compared to the GDP and when the duration of the negative effects is small. The last assumption is that the economic loss is treated as an exogenous force, which is independent from the GDP. This assumption seems to be valid if the economic loss associated to a structural failure is small compared to the GDP. Then the loss can be treated as constant and independent of the GDP. For the case of the WTC failure however, the failures of the towers were triggered by terrorist attacks and this had also influence on the consumer and investor confidence.
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