Landfill Slope Stability Risk Assessment

by

M. Ali Jahanfar

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ABSTRACT

Landfill Slope Stability Risk Assessment

M.A. Jahanfar  
University of Guelph, 2014  
Advisor:  
Professor Brajesh Dubey

Landfills have consistently, in the past as well as today, been the primary method for waste disposal. In the past 20 years, catastrophic landfill slope failures have occurred worldwide, especially the fatal ones. For instance, Bandung (2005) and Payatas (2000) dumpsite failures killed 147 and 300 people, respectively. This calls for more landfill failure investigation to prevent loss of human life and major environmental consequences.

The goal of this study, therefore, is to introduce the framework to evaluate the risk of landfill slope failure. Simply put, risk is equal to the hazard of the probable landfill slope failure multiplied by the consequence of the failure (e.g., waste material flow over the residential area). Landfill failure hazard is a function of waste material geotechnical properties, while the consequence is a function of waste material rheological parameters.

The range of geotechnical and rheological characteristics of the waste materials are provided by analyses of documented landfill and dumpsite failures. In terms of geotechnical characteristics, the mean values of unit weight, friction angle and cohesion are obtained 11.1 kN/m³, 25.4° and 10 kPa, respectively. For rheological feature, four classes of friction global distributions were recommended for run-out analysis of waste-flow given the potential for landfill failure. These results can be used to estimate the risk of existing landfills, manage them, and prevent the catastrophic disasters.
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Chapter 1
Introduction

1.1 Overview of the Problem

The development of waste recycling and recovering techniques as well as new treatment methods cannot overcome the landfill requirement to accommodate the residual waste. For example, although North America contains 5% of the world’s population; this region is the largest Municipal Solid Waste (MSW) producer (UNEP, 2012). Based on new data from the USEPA, the US produced around 250 million tons solid waste in 2012, which is 2.8 times more than the amount produced in 1960. After 65 million tons of recovered and 20 million tons of composted waste, the rest of MSW (165 million tons or 66%) were buried in the landfills. with the engineered ways of the compaction and landflling (12 kN/m$^3$ unit weight), 135 km$^3$ landfill volume is needed to bury this vast amount of the waste. In recent years, siting a new landfill is becoming a major problem in North America. As part of the NIMBY (Not In My Back Yard) attitude, there is pressure on existing landfills to accommodate larger amount of waste. Maximizing the landfill capacity directly depends on the landfill geometry by increasing the height and slope inclination. The major concern in this regard is the landfill stability (Singh and Murphy, 1990).

Till the mid-1980s, not much attention had been given to the subject of stability in landfills. In 1984 a landfill in North America failed, which caused the movement of 110,000 m$^3$ of waste. Following this failure and many others in the coming years, more and more research was conducted on the stability of landfills (Stark, 1999; Koerner and Snoog, 2000; Koelsch, 2007; Blight, 2008; Gharabaghi et al., 2008).
The Factor of Safety (FOS) is defined as the ratio of resisting forces plus moments as numerator to driving forces plus moments as denominator. Although the safety of natural and artificial slopes contain uncertainties, it is common to use the same FOS for different degrees of uncertainty involved in its calculation in order to assess slope stability; it is not logical to use the same criteria for such a widely varying degree of uncertainties (Duncan, 2000). This approach is called deterministic in comparison with a probabilistic approach, which provides the mean of the uncertain parameters and distinguishes a factor between high and low uncertain conditions (Lacasse et al., 2007).

Risk assessment and hazard evaluation of the slopes have been allowed by probabilistic approach (Christain, 1994; Lacasse, 2007; Chi et al., 2011). However, the risk has not commonly been accounted for in geotechnical analyses in landfills. Lacasse et al. (2007) defined risk as a measure of the adverse effects on probability of life, health, property, or environment. Simply put, risk is equal to the hazard of the probable event multiplied by the potential worth of loss (consequence). The main benefit of risk assessment is that all cause and effect relationships associated with slope instability can be incorporated. In other words, it facilitates a structured approach to understand slope failure processes and the consequences. This is while the conventional analysis does not include the consequences, and emphasizes merely the slope failure process. Recently, the concept of risk assessment has been developed for slope stability evaluations. The qualitative risk assessment has been used since the 1970’s (Varnes, 1984; Whitman, 1984; Einstein, 1988), and extended to quantitative methods in the 1990’s (Fell and Hartford, 1997; Nadim and Lacasse, 1999; Ho, et al., 2000; Nadim and Lacasse, 2004; Hartford and Baecher, 2004; Lee and Jones, 2004).

Landfills are one of the largest man-made potential sources of contamination which also involve various stability issues. Landfill slope stability requires risk assessment to evaluate the hazard and consequence of the failure. The hazard will refer to failure probability and the annual frequency of trigger mechanisms such as rainfall. The consequence would be the failure exposure on the environment, human life, and economic loss.
1.2 Scope

This thesis:

1. Identifies and evaluates major parameters (unit weight, friction angle, cohesion) of landfill materials in terms of slope stability.

2. Describes the six landfill failure case studies.

3. Presents the back-analysis of the cases to evaluate landfill slope stability using Slope/W software (Krahn, 2004).

4. Presents the back-analysis of the cases to evaluate waste mobility after landfill failure using DAN-W software (Hungr, 2010).

5. Uses slope stability back-analysis results to allocate the best statistical distributions for major parameters of Municipal Solid waste (MSW).

6. Uses mobility back-analysis results to choose the best statistical distribution for waste mobility parameters.

7. Describes the potential failure case study, using existing landfill data from Delhi, India, and evaluates the probability of failure for different failure scenarios (Hazard), and the vulnerability as a result of waste movement after the failure (Consequence). In the end, it assesses the risk from the case study.

The objective of this study includes slope stability and waste run-out assessment of landfills. In terms of slope stability of landfills, the task is to consider a probabilistic approach and provide the global distributions for unit weight, friction angle, and cohesion of waste materials. The second task is the waste run-out evaluation of landfills which offers appropriate probabilistic distributions of the frictional parameters for waste movement during the landfill failure process. Evaluating the probability of failure (Hazard) by the results of first task and assessing the probability of life loss (Consequence) by the results of second task, the eventual aim is to calculate the risk of landfill failure (an example case study is presented). The evaluation
of risk may help to suggest safe height and slopes for existing landfills and prevent catastrophic landfill failures.

### 1.3 Organization of the Thesis

Chapter 2, 3 and 4 in this study have been organized in manuscript format. The second chapter presents the previous efforts (literature-review) on the waste material shear strength and unit weight. To continue, these parameters are improved based on six back-analyses using Slope/W software and the Monte-Carlo method. This chapter concludes with the calculation of probabilistic distributions of geotechnical characteristics of MSW from historical failure sites. Chapter 3 presents the back-analyses on five failed landfills with DAN-W software using post-failure characteristics to obtain rheology parameters of these sites. The result of this study is the global probabilistic distribution for rheology parameters. Chapter four used stability back-analyses results (from Chapter 2) to insert input data for desired landfill and evaluate the probability of failure for different failure scenarios (Hazard). The results of the mobility back analyses (from Chapter 3) are applied to evaluate travel distance of the waste materials after the failure for different scenarios and eventually expected fatalities. The landfill risk evaluation is the last part of the Chapter 4. Chapter 5 contains the conclusions and recommendations for future research.
Chapter 2
Landfill Slope Stability Analysis

2.1 Introduction

Landfills have consistently, in the past as well as today, been the primary method for waste disposal. The negligence of humans towards solid waste has fortunately started to fade away since early in the 20th century. Since then, solid waste production grew in both quantity and variety of sorts, and as a result landfill siting became a major environmental issue. Yet, not until two decades ago, did the stability of these ever-expanding landfills receive any attention. In the past 20 years, catastrophic landfill slope failures have occurred worldwide (e.g., Bandung dumpsite in Indonesia and Payatas landfill in the Philippines) indicating that more landfill stability investigations and analysis needs to be done to prevent loss of human life and major environmental, operational and economic burdens. North America, too, has had its share of landfill slope failures (e.g., Rumpke, Kettleman and Maine). Fortunately, these failures have not caused any life loss, but have caused serious environmental problems such as intrusion of landfill waste to lower grounds, blockage of roads, and contamination of downstream rivers due to leachate, which interrupts and impacts the life of thousands of nearby residents (Seed et al., 1990; Stark and Eid, 2000; Koerner and Snoog, 2000).

Landfill failures have occurred during the landfill construction, operating time or after closure. The failure surfaces were mostly within the waste materials or soil-geosynthetic interface (Omari, 2012). These failures probably cause catastrophic results including life loss, property loss, and environmental pollution including that of surface and groundwater. Therefore,
the geotechnical engineers are facing with challenges of landfill stability on one side while maximizing the landfills capacity on the other side. In this chapter, landfill failure modes are reviewed followed by the denotation of the stability factors. In the subsequent sections, the failure mechanisms and historical landfill failures are studied. The last section presents the computer codes which have been used to model the landfill failures.

2.1.1 Landfill Failure Modes

Because of the various consequences of different kinds of landfill failures, it is important to study the ways of the failure to control and manage landfill stability. Following the previous landfill failures more and more research has been conducted on this subject. Stark (1999) classified landfill failure into two predominant modes, translational and rotational. Based on previous experiences, translational failure is probably more catastrophic and common rather than rotational failure. Translational failure mostly starts with a huge sliding mass and then deforms and breaks into several independent pieces (Stark, 1999). The translational failure may start from the weak surfaces within the waste materials or waste-liner interface. The weak surfaces within the waste materials can be the ones separating the waste from poor-wealthy neighbourhood or summer-winter layers.

Figure 2-1 shows the possible failure paths due to the seasonal changes in waste composition in landfill (Blight, 2008). Different shear strength between waste components and liners can justify the translational failure in waste-liner interface.

![Figure 2-1: Seasonal changes in waste composition in landfill (S=summer, W=winter), possible failure paths are AAA and BBAA (Blight, 2008)](image-url)

Stark (1999) claimed that homogeneous materials such as soft soil and clay are more involved with rotational mode of failure and translational failure is more possible in landfills. However, one year after his published research, the tragic rotational failure happened in Payatas landfill in Philippine with more than 1.2 million m$^3$ volume of the slipped waste leading to the death of 300 people. This accident highlighted the necessity of more research on this mode of failure. Thus, a year after, Qian et al. (2001) published the general landfill failure modes. Figure 2-2 illustrated six main modes of the landfill failure within the cover liner, waste materials and foundation soil or their interfaces.

Figure 2-2: Landfill failure modes (Qian et al, 2001)
The initial two failures are called veneer failures, and although highly important, can be avoided simply by analysis and rectified at a reasonable cost if failure occurs. The other modes of failure are more severe in nature. Specifically, failures of the types (d) and (f) are the most critical and cases of massive failures that have occurred within landfills at gigantic costs. Type (d) is rotational failure along the waste and foundation soil, while type (f) is translational failure above, beneath or within the liner system at the base of the landfill which can extend through the waste or continue along the liner system in back slope.

2.1.2 Factors Affecting Landfill Stability

Pore Water Pressure

Excessive pore water pressure is the most emphasized failure reason in landfill stability (Merry et al., 2005; Koerner and Snoog, 2000; Bauer et al., 2008; Blight, 2008; koelsch, 2005). Elevating the water level will cause excessive pore water pressure and consequently reduce the effective stress and mobilized shear strength. Water level elevation can happen due to:

1. Heavy rainfall: 10 days of rainfall before Payatas, Philippine; 3 days of rainfall before Bandung, Indonesia; failures are the trigger mechanisms of landfill failure due to excessive pore water pressure (Bauer et al. 2008).
2. Leachate build up: Due to the drainage problems or inappropriate recirculation, leachate builds up in landfill which is classified as a liquid related trigger mechanism of the landfill failure by Koerner and Snoog (2000).
3. Landfill gas builds up: Merry et al. (2005) believed that waste biodegradation may produce trapped gas in landfill and elevate pore water pressure in high level of waste saturation.
4. Landfilling plastic bags: Surprisingly, Bogota, Colombia landfill failure (1997) leads to the conclusion that plastic bag sheets on the waste bodies in the landfill layers may “retain water and affect shear strength” (Blight, 2008). Using plastic bags led to higher horizontal permeability rather than vertical one resulting in retaining water. In addition, shear strength could potentially reduce by many plastic-to-plastic interfaces in the waste.
A practical way of calculating pore water pressure on slip surface is pore pressure ratio \( R_u \) which is equal to:

\[
R_u = \frac{\gamma_{water} \times h_{water}}{\gamma_{waste} \times h_{waste}}
\]  

[2-1]

where \( \gamma_{water} \) is the unit weight of water; \( h_{water} \) is the height of the water on slip surface; \( \gamma_{waste} \) is the unit weight of the MSW; \( h_{waste} \) is the height of the MSW from slip surface to the top of the landfill.

Field-measured pore water pressure ratio at a saturated slope of waste at the Brock West Landfill site in Ontario has shown that \( r_u \) value is approximately equal to 0.2 for stable slope (Dewaele et al., 2005).

**Geometry**

Height and side slope are the main driving geometrical parameters of the landfill which affect stability (Omari, 2012). Basically, the shear strength of the waste materials depends on site geometry. Landfill failure will occur when slope and height are constructed higher than the tolerable amount (Koelsch, 2007). For example, the US landfill failure (Rampke, 1996) happened when the landfill height exceeded the tolerable elevation by 13 to 15 meter (Stark and Eid, 2000). Also, landfill slope around the failure zone in the Payatas, Philippine landfill was estimated 1.5 H: 1V (Merry et al., 2005). Generally, the lack of design in dumpsites and the lack of design understanding in engineered landfills are evident in waste failure by additional height and steep side slope.

**Waste Shear Strength**

The shear strength of all materials largely depends on the corresponding strain. This phenomenon is called “strain-softening”. Simply, the material will deform as a result of the load increase. More loads provide more deformation until the resistance reaches the “peak value of the strength”. After the peak point, the deformation increases while the material resistance remains constant or drops (Boutwell, 2004). Figure 2-3 demonstrates this theory.
The shear strength increased with displacement growth because MSW has been reinforced by plastics, ropes, fabrics and similar textures (Stark et al., 2009). Nevertheless, Blight and Fourie (2003) believed that Istanbul, Bogota, Durban, and Payatas landfill failures are the “Strain-Softening” cases of waste materials. Applying Strain-Softening theory for waste, Boutwell (2004) compared this feature for waste, soil, and plastic (geo-synthetic) materials (Figure 2-4).

Boutwell (2004) explains that if geo-synthetic reaches to the peak resistance of 0.1 inch of deformation, waste materials is at 1% to 5% of its strength, and soil resistance probably is at the half of the peak strength.
Figure 2-5: Stress–displacement relationships from direct shear tests on MSW. Numbers in parenthesis are the testing
normal stresses in kPa. (Stark et al., 2009)

Due to MSW heterogeneity and fibrous texture, MSW characteristics changes with age and degradation making it extremely hard to prepare undisturbed sample. Stark et al. (2009) published the shear stress and displacement relationship compiling results of several research studies (Figure 2-5).

Waste density and type are the most effective parameters on the waste shear strength. Koelsch (2007) classified waste density as a major reason for landfill failure in Payatas, Philippine (2000). In principle, the absence of waste compaction reduces the rainfall surface flow, and increases the rate of water percolation into the waste. The infiltrated water mobilizes the excessive pore water pressure, and reduces the waste shear strength. Blight (2008) examined that the high waste compaction can lead to high-strength cohesive waste material and vice versa. However, Koelsch (2007) emphasized that the major reason of the low waste density in Payatas failure was waste composition. He explained that scavenging of the waste materials in Payatas landfill had completely changed the waste composition. High strength materials such as wood, metal, cardboard, intact bottles were segregated, leaving the waste with organic and light plastic portions. The inappropriate composition of waste is not limited to the developing countries; for example, at the Rumpke landfill failure in US (1996), the failure has happened due to the light waste combination and wrong leachate circulation (Koelsch, 2007).
As mentioned in previous section, the dump which contains poor and wealthy community wastes is potentially exposed to translational failure. Table 2-1 showed the example of different waste compositions based on financial status (Blight, 2008).

<table>
<thead>
<tr>
<th>Component of Waste</th>
<th>Poor Community</th>
<th>Wealthy community</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ash, Dust and Sweeping</td>
<td>66%</td>
<td>Negligible</td>
</tr>
<tr>
<td>Paper, Cardboard and Textiles</td>
<td>4%</td>
<td>25%</td>
</tr>
<tr>
<td>Plastic</td>
<td>3%</td>
<td>12%</td>
</tr>
<tr>
<td>Metals</td>
<td>1%</td>
<td>10%</td>
</tr>
<tr>
<td>Glass</td>
<td>1%</td>
<td>9%</td>
</tr>
<tr>
<td>Food and Garden waste</td>
<td>20%</td>
<td>41%</td>
</tr>
<tr>
<td>Miscellaneous other</td>
<td>5%</td>
<td>3%</td>
</tr>
</tbody>
</table>

The biggest gap between poor community waste and wealthy community waste is the large content of ash, dust and sweeping in poor community, while these components are negligible in wealthy community. Blight (2008) also mentioned that at a same period, a poor community produces two to four times more waste as compared to a wealthy community. Therefore, it is reasonable if thick layers of poor waste and thin layers of wealthy waste constitute the dump layers, one by one. Similar situation can happen in seasonal changes (Figure 2-1).

**Liner Shear Strength**

The landfill liners are different on their types, layers and operation. The most straightforward classification leads to Compacted Clay Liner (CCL) and Geo-synthetic Clay Liner (GCL). As mentioned above, the peak value of shear strength for plastic (Geo-synthetic) and soil is dramatically lower than waste materials. This fact increases the probability of translational failure in comparison with rotational failure in landfills. For example, the Kettleman, US landfill failure in 1988 occurred over the length of the base layer due to “insufficient strength of geo-synthetic” (Adamczyk, 2005). In the other study, Koerner and Snoog (2000) concluded that “wet clay beneath the geo-membrane” and “excessively wet foundation soil” are the liquid related trigger mechanisms of landfill failures.
Climate

The effect of regional climate on landfill stability is a disputable issue in two ways:

1. Tropical countries that experience landfill failure (Philippine, Indonesia) are highly involved with heavy rainfall in wet seasons and fire problems in the dry seasons. The Bandung, Indonesia landfill failure (2005) occurred due to heavy nonstop rain for 3 days and the previous landfill fire problems (Bauer et al., 2008).

2. Cold regions could potentially suffer from freezing and throwing. This phenomenon affects landfill stability (Omari, 2012). The freezing of slope toe and throwing the slope summit will increase the pore water pressure and reduce the effective shear strength. In addition, freeze-thaw can destructively affect the hydraulic conductivity of the compacted clay. However, there is no record for landfill failure with direct effect of freezing and thawing phenomenon.

2.1.3 Historical Landfill Failures

Landfill or dumpsites failure study is a relatively new concept. It may be due to the fact that the landfill sizes were not big enough to cause failure, and the failures until recently were not fatal (Blight & Fourie, 2003). In many of these failures no scientific investigation was carried out to ascertain the cause of failure. Mostly, the governments do not report such events due to the social unacceptability of the issue. However, many significant landfill slope failures have occurred recently, especially in high precipitation countries such as Philippines, Indonesia and even the US. These failures have caused grave human loss, as well as economic and environmental burdens of huge proportions on local governments and nearby residents and workers. Table 2-2 illustrated such cases.
<table>
<thead>
<tr>
<th>No.</th>
<th>Reported by</th>
<th>Region</th>
<th>Year</th>
<th>Waste Vo.</th>
<th>Failure Mode</th>
<th>Life Loss</th>
<th>Reason</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Gillani, 2013</td>
<td>Iran, Shiraz</td>
<td>2013</td>
<td>N/A</td>
<td>Rotation</td>
<td>11</td>
<td>The firework on the burned waste materials in landfill, caused exceeding pore water pressure and collapse</td>
</tr>
<tr>
<td>2</td>
<td>Blight, 2008</td>
<td>Indonesia, Bandung</td>
<td>2005</td>
<td>2.7 million</td>
<td>Translation</td>
<td>147</td>
<td>The reason is the extraordinary large portion of light waste (plastics) combined with disturbed water balances due to leachate circulation. 10 days Heavy rain, Low waste density, Water percolation instead of drainage, reducing waste shear strength</td>
</tr>
<tr>
<td>3</td>
<td>Blight, 2008</td>
<td>Philippines, Payatas</td>
<td>2000</td>
<td>1.2 million</td>
<td>Rotational</td>
<td>330</td>
<td>The increase of leachate head behind the clay backfill trench, due to the presence of both the clay layer within the waste mass and the clayey backfill material within the trench, was most likely the triggering mechanism of the failure. The failure occurred after 48-hour of rainfall. The triggering mechanism was felt to be excessive liquid waste placement into the already-saturated woodland bark between the old and the recent sections of the landfill. The increase in leachate head within the waste mass due to the aggressive leachate injection operations.</td>
</tr>
<tr>
<td>4</td>
<td>Koerner and Snoog, 2000</td>
<td>US</td>
<td>1997</td>
<td>100,000</td>
<td>Rotation</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>5</td>
<td>Koerner and Snoog, 2000</td>
<td>Africa</td>
<td>1997</td>
<td>300,000</td>
<td>Translation</td>
<td>-</td>
<td>Excessive leachate level buildup (estimated to be 5 m) within the old, decomposed waste caused by water infiltrating from adjacent surface water ponds</td>
</tr>
<tr>
<td>6</td>
<td>Koerner and Snoog, 2000</td>
<td>South America</td>
<td>1997</td>
<td>1.2 million</td>
<td>Translation</td>
<td>-</td>
<td>The additional buildup of leachate head in the landfill due to ice formation at the exposed waste face near the toe of the slope</td>
</tr>
<tr>
<td>7</td>
<td>Blight, 2008</td>
<td>Colombia, Bogota</td>
<td>1997</td>
<td>800,000</td>
<td>Translation</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>8</td>
<td>Blight, 2008</td>
<td>South Africa, Durban</td>
<td>1997</td>
<td>160,000</td>
<td>Rotation</td>
<td>-</td>
<td>Excessive leachate level buildup (estimated to be 5 m) within the old, decomposed waste caused by water infiltrating from adjacent surface water ponds</td>
</tr>
<tr>
<td>9</td>
<td>Koerner and Snoog, 2000</td>
<td>US, Mahoning</td>
<td>1996</td>
<td>100,000</td>
<td>Translation</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>10</td>
<td>Koerner and Snoog, 2000</td>
<td>US, Rumpke</td>
<td>1996</td>
<td>1.2 million</td>
<td>Translation</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>11</td>
<td>Koerner and Snoog, 2000</td>
<td>Europe</td>
<td>1994</td>
<td>60,000</td>
<td>Translation</td>
<td>-</td>
<td>Excessive leachate level buildup (estimated to be 5 m) within the old, decomposed waste caused by water infiltrating from adjacent surface water ponds</td>
</tr>
<tr>
<td>12</td>
<td>Koerner and Snoog, 2000</td>
<td>Turkey, Umbanye</td>
<td>1994</td>
<td>12,000</td>
<td>Translation</td>
<td>39</td>
<td>The additional buildup of leachate head in the landfill due to ice formation at the exposed waste face near the toe of the slope</td>
</tr>
<tr>
<td>13</td>
<td>Koerner and Snoog, 2000</td>
<td>US, Maine</td>
<td>1989</td>
<td>500,000</td>
<td>Rotation</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>14</td>
<td>Koerner and Snoog, 2000</td>
<td>US, Kettleman</td>
<td>1988</td>
<td>490,000</td>
<td>Translation</td>
<td>-</td>
<td>Excessive leachate level buildup (estimated to be 5 m) within the old, decomposed waste caused by water infiltrating from adjacent surface water ponds</td>
</tr>
<tr>
<td>15</td>
<td>Koerner and Snoog, 2000</td>
<td>US</td>
<td>1984</td>
<td>110,000</td>
<td>Rotational</td>
<td>-</td>
<td>Excessive wetness of the clay component of the HDPE geo-membrane to CCL interfaces. It was reported that the geo-membrane was placed during a very wet period when the CCL was already at high water content.</td>
</tr>
<tr>
<td>16</td>
<td>Blight, 2008</td>
<td>Yugoslavia, Sarajevo</td>
<td>1977</td>
<td>200,000</td>
<td>Translation</td>
<td>N/A</td>
<td>Excessive leachate level buildup (estimated to be 5 m) within the old, decomposed waste caused by water infiltrating from adjacent surface water ponds</td>
</tr>
</tbody>
</table>

2.1.4 Methods of Slope Stability Analyses

Generally, there are two kinds of analyses to determine the stability of geotechnical structures: limit equilibrium and finite element analysis. Finite element analysis is based on stress and deformation, while limit equilibrium is the force and moment basis. The waste stress-strain parameters for finite element analysis are hardly accessible in comparison with strength parameters of limit equilibrium ones. In addition, most of the slope stability methods have been
focused on limit equilibrium (Omari, 2012). Factor of safety is the outcome of this analysis as a ratio of resisting force and moment summation to the driving force and moment summation:

\[
FOS = \frac{\sum \text{Resisting forces,moments}}{\sum \text{Driving forces,moments}}
\]  

[2-2]

A factor of safety with the value lower than one indicates that slope failure will happen, while the higher values would be acceptable in terms of safety. However, all depend on the data accuracy, which is doubtful in geotechnical structures especially for landfills and dumps. Thus, it is common to use FOS=1.5 as a reliable value for slope stability in geotechnical projects (U.S Army Corps of Engineers, 1997).

The slices method by far is the most practical limit equilibrium method to analyse slope stability. In this method, the weight of the mass above the slide surface mainly affects the normal stress on this surface (Lambe and Whitman, 1969). To calculate the factor of safety, the sliding mass has to be divided to several vertical slices and the equilibrium of each slice should be determined in terms of force and moments. This process has to be repeated many times to find the minimum factor of safety, which belongs to the most critical slide mass in the slope.

Different assumptions for inter-slice forces brought different solutions (ordinary, simplified Bishop, Janbu simplified, Spencer and Morgenstern-Price) for the method of slices. Nevertheless, the minimum factor of safety in all solutions is slightly varied by 10% except the ordinary solution which can be varied as much as 60% (Whitman and Bailey, 1967). Table 2-3 illustrated different assumptions, forces or moments which are used in all solutions of slices method.

<table>
<thead>
<tr>
<th>Method</th>
<th>Factor of Safety (FOS)</th>
<th>Inter-slice force assessment (H=Horizontal, V=Vertical)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force equilibrium</td>
<td>Moment equilibrium</td>
</tr>
<tr>
<td>Ordinary</td>
<td>-</td>
<td>Yes</td>
</tr>
<tr>
<td>Bishop’s Simplified</td>
<td>-</td>
<td>Yes</td>
</tr>
<tr>
<td>Janbu’s Simplified</td>
<td>Yes</td>
<td>-</td>
</tr>
<tr>
<td>Janbu’s Generalised</td>
<td>Yes</td>
<td>-</td>
</tr>
<tr>
<td>Spencer</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Morgenstern-Price</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Lowe-Karafiath</td>
<td>Yes</td>
<td>-</td>
</tr>
<tr>
<td>Corps of Engineers</td>
<td>Yes</td>
<td>-</td>
</tr>
</tbody>
</table>
Describing the shear strength of the slope materials with cohesion and internal friction angle, all of the tabulated methods in Table 2-3, are useful for slope stability analysis. However, Bauer et al. (2008) introduced an advanced version of the Bishop Method to consider the reinforcement effects on landfill stability calculation. Waste materials contain tensile forces (because of fibers and foils) which cause reinforcement effects. Bandung and Bandeirantes landfill failures have been back-analysed by Bauer et al. (2008) with both conventional and advanced Bishop Methods. The results proved that the conventional method underestimates factor of safety in comparison with advanced method. Although, the factor of safety underestimation in design is an advantage, it may lead to wrong strength values for waste characteristics in back-analysis (Bauer et al., 2008).

**Two-dimensional and Three-dimensional Analyses**

Assuming 2-dimension rather than 3-dimension, the slope stability problems are simplified in driving and resisting forces. To compare these two perspectives, Boutwell (2004) oversimplified the situation as it is shown in Figure 2-6.

![Figure 2-6: Oversimplified comparison between 2D and 3D slices method (Boutwell, 2004)](image)

The weight of the block contains the driving force while the sliding surface is involved with shear strength as a resisting force. The critical value of block height \( H_c \) would be equal to:

\[
H_c = \left( \frac{4c}{\gamma} \right) \times \left( 1 + \frac{H_c}{2B} \right), \tag{2-3}
\]

where \( \gamma \) is the soil unit weight, \( c \) is the soil cohesion and \( B \) is the length of block along face.

Rearranging equation [2-3] in terms of factor of safety for 2D and 3D conditions:
\[ FOS_{3D} = FOS_{2D} \times \left(1 + \left(\frac{H_C}{2B}\right)\right) \]  \hspace{1cm} [2-4]

It means that the wedge factor of the typical landfill with 80 meter height and 300 meter length, would be around 1.13.

Koerner and Snoog (2000) studied 10 landfill failures with both 2D and 3D analysis which confirmed that the wedge factors are calculated between 1.06 and 1.25. He inferred that the elimination of the side shear forces in 2D analysis was the major reason of conservative results with this method. Although, a 2D slope is simpler and more conservative than 3D stability analysis, it is impossible to substitute 2D for 3D slope stability analysis in literature. Stark (1999) concluded that 2D analysis is appropriate for slope designs due to the conservative results, but 3D analysis can potentially be used for back-analysis to find the real shear strength along the slide surface. Rumpke landfill failure is modeled with 3D and 2D back-analysis. The results indicated that the shear strength in 2D analysis is obtained more conservative than 3D by 25% to 30% (stark and Eid, 2000). In addition, Yu and Batlle (2011) emphasized that 2D analysis is not helpful for some spatial situations such as: slope with lateral or point loads, lateral complex geometry and piezometric level, and horizontally anisotropic and heterogeneous materials, etc. Yu and Batlle (2011) used quasi-three-dimensional slope stability analysis to determine the potential slide mass in Spanish landfill. Equivalent 3D factor of safety in this method has been obtained from individual results of 2D analyses with 44 parallel cross sections within the potential sliding mass. Within this study, the equivalent 3D results have been compared with 2D analyses of the steepest slope cross section (Figure 2-7).
Trying various combinations of cohesion (0 kPa to 60 kPa) and internal friction angle (10° to 40°), Yu and Battle (2011) determined the 2D and 3D factor of safety (Figure 2-7). The results indicated that 3D factor of safety is higher than 2D factor of safety by 46% to 87%.

However, in the other study, Adamczyk (2005) analysed the Kettleman landfill by 2D and 3D back-calculations. As a result, he figured out that 2D factor of safety is significantly higher than 3D one. This author argued that these incompatible results may be derived from the real geometry reproduction by 3D analysis in this study. The results also indicated that 2D analysis underestimated the internal friction angle in comparison with 3D analysis ($\Phi = 4.3^\circ$ for $FOS_{2D}$ and $\Phi = 8^\circ$ for $FOS_{3D}$).

**Computer Software Use in Stability Analyses**

There are various kinds of software available to analyse the slope stability. These software (Slope/W, CLARA, FLAC, UTEXASED, etc.) classify as limit equilibrium or finite element, and 2D or 3D or both. These computer codes can be used to predict the stability based on input data and the amount of factor of safety for considering slope. In addition, back-analysis is the favourable method to determine operable material parameters. During the recent decade, computer codes have been used for landfill stability design and failure back-analysis. However,
to understand the scientific ways of using these programs, the literature of landfill slope stability modeling methods are reviewed in this section.

Koerner and Snoog (2000) back-analysed ten landfill failures which are divided to unlined (or soil lined) and lined landfills with geo-synthetic materials. In all cases 3D and 2D Geo-studio software have been used to model the landfills using Bishop and Janbu methods. Firstly, shear strength parameters of the waste materials have been back-analysed by a 3D computer code to reach FOS=1. Secondly, the trigger mechanisms, unique to each failure, applied to the model causing FOS<1. Finally, the obtained shear strength as well as trigger mechanisms were supplied in 2D analysis. To model the trigger mechanisms, different methodologies were applied for different events. For example, heavy rainfall and leachate head increase were modeled with raise in phreatic surface, and wetness geo-membrane-clay interface were simulated with shear strength decrease along the interface. The results showed liquid-related trigger mechanisms in all failures.

In the same year, Stark and Eid (2000) modeled the Rumpke landfill failure (1996). The simplified Janbu method in CLARA slope stability software was used for this simulation. Combining 18 parallel cross sections, the program interpolated between 2D cross sections to provide 3D geometry. Because of translational failure along the liner, a back-analysis of waste mass failure was conducted to evaluate the shear strength of the liner. Grounded on borehole results and observations, the maximum leachate level prior to the failure was assumed. However, sensitivity analysis was utilized to show the shear strength changes in different leachate levels, while the other parameters were constant.

Adamczyk (2005) analysed the Kettleman landfill failure (1988). A model was taken from this failure using FLAC and 3DFLAC for 2D and 3D analysis, respectively. The author reduced the shear value along the waste and subgrade as a starting mechanism of the failure. Shear strength reduction technique provided lower value of internal friction angle for 2D analysis rather than 3D in order to reach FOS=1. It should be noted that the effect of pore water pressure in landfill has been neglected in Adamczyk (2005) study.

The landfill near Great Lake area in US failed due to excessive soil stockpile on the top. This rotational failure was analysed with 2D Slope/W program using Bishop Method (Yao and
Tsai, 2006). Because of rotational failure through the foundation, the shear strength of this layer was reduced by 30% as a trigger mechanism of the landfill failure. Shear strength of the waste materials was assumed bi-linear which is suggested by Kavazanjian et al. (1995). The results indicated FOS<1 due to the foundation instability by less shear strength.

The most fatal landfill failure happened in Payatas, Philippine (2000) with more than 300 victims. Merry et al. (2005) calculated the stability of this landfill with UTEXASED software using Spenser’s Method. To predict the saturated depth of the waste, three months of precipitation records were quantified by the HELP (Hydraulic Evaluation of Landfill Performance) software. In addition, regarding the waste saturation and continuous degradation, Merry et al. (2005) inferred that landfill gas could not migrate freely; this fact will affect the normal unit weight of the water. Therefore, increasing the water unit weight to 21 kN/m$^3$, the factor of safety was lowered to one and the failure situation was created.

Two Brazilian dumpsites at the Muribeca site were compared to the failed Maceio site were compared in terms of slope stability (Gharabaghi et al., 2008). Geo-studio software was used to model these landfills side slopes with Morgenstern-Price method. Tri-linear shear strength, which is the three different shear strengths in three different heights of the waste materials, was enveloped to model the waste material geotechnical characteristics. Operating sensitivity analysis on different pore pressure ratios (R_u), safety factor was obtained. The results confirmed a lower factor of safety for the failed landfill (Maceio site) in comparison with the stable one (Muribeca site).

Two case studies of landfill failures, Bandung and Bandeirantes, are simulated to compare two different slope stability calculation methods: conventional Bishop Method and advanced Bishop Method (Bauer et al. 2008). The factor of safety for Bandung and Bandeirantes landfill failures calculated with both methods. Based on the results, the second method, which considers the reinforcement effects of the waste materials, indicated a higher factor of safety. Sensitivity analysis had been conducted for water level inside the landfill and internal friction angle of the waste materials. Obviously, a higher level of water leads to a lower safety factor.
2.2 Objective

Although literature contains precious studies on historical landfill failures, they mostly employ deterministic methods rather than probabilistic methods. The factor of safety, which is used in conventional geotechnical practice, is not a function of the uncertainty degree of waste geotechnical characteristics. In other words, similar values of factor of safety are the criteria for various degrees of uncertainty, which is not a logical approach. Probability and reliability calculations provide meaningful discrimination between low and high degrees of uncertainty in the geotechnical approach (Duncan, 2000). The objective of this study is to compare different landfill failures considering the uncertainty of unit weight and shear strength of the waste materials, and provide the best fit statistical distributions for landfill slope stability parameters.

2.3 Methodology

In order to analyse the landfill slope stability, Slope/W is used, as it has been applied in the earlier research (Koerner and Soong, 2000; Yao and Tsai, 2006; Gharabaghi et al., 2007). Slope/W program uses limit equilibrium theory to calculate the factor of safety (Krahn, 2004). Based on the high uncertainty of geotechnical characteristics of the waste materials, the input parameters for these materials are assigned a probabilistic status. This software can provide Monte-Carlo probabilistic analysis for these sorts of input data. Using probabilistic input data with Monte-Carlo, the probability distribution of factor of safety is computed.

To find probabilistic input data, this research reviews relevant studies (as summarized below) on MSW characteristics and introduced statistical distributions for each one. These primary distributions have been used as inputs to the back-analysis.

There are three dumpsite failures (Sarajevo, Istanbul and Bandung) and three landfill failures (Ohio, Bogota and Durban) which are described herein. The pre-found primary distributions of unit weight, friction angle, and cohesion of the waste materials, as well as pore water pressure ratio for each case have been used as inputs to the Slope/W software with Monte-Carlo method to obtain the factor of safety distribution. Considering the fact that, these slopes have already failed, back-analysis of slope failures are carried out to improve primary
distributions of unit weight, cohesion, and friction angle of waste materials at the moment of slope failure. To approach this goal, “simplified probabilistic back-analysis” has been suggested by Zhang et al. (2011). Zhang et al. (2011) described the mathematical equations to back-calculate the normally distributed parameters until FOS=1±0.05 (for details see Appendix A).

Figure 2-8 illustrates the flowchart of the methodology. Using literature data as prior distributions for the slope stability parameters (unit weight, friction angle, cohesion and pore pressure ratio), three dumpsite failures and three landfill failures are back-analysed. Continuing the loop (Figure 2-8), until FOS=1±0.05, the optimized probabilistic distributions for each parameter was obtained. These global distributions can be utilized for project-specific design when data are not available.
2.4 Results

2.4.1 Unit Weight Initial Distribution (Literature-data)

The unit weight of MSW is a significant parameter in stability analysis of landfill slopes, but important uncertainty currently exists regarding its value (Zekkos et al., 2006). As with soil, the unit weight of the waste materials depends on compaction, depth, and the amount of moisture. Unlike soil, unit weight of the waste materials contains highly variable components. In addition, the unit weight of the waste materials varies with age due to degradation. Furthermore, the process of testing the unit weight of the waste materials involves some specific difficulties, which are summarised by Fassett et al. (1994) as follows: distinct waste components from daily soil cover, spatial and temporal assessment of the unit weight, and sampling difficulties in depth. The previous efforts for unit weight measurement of the waste materials have been illustrated Figure 2-9(a). According to the normality tests such as Kolmogorov-Smirnov and Shapiro-Wilk (at the 5% level of statistical significance), the unit weight data do not show significant evidence of non-normality. Thus, the normal distribution of the unit weight histogram is plotted with 10.9 kN/m$^3$ and 3.7 kN/m$^3$ as the mean and standard deviation values, respectively. In addition, as illustrated in Figure 2-9(b), the data plot as a relatively straight line, confirming that the data is adequately described by the normal distribution.

Figure 2-9: (a) Unit weight data histogram (b) Probability plot for normality
There is insufficient information on unit weight range in the technical literature. Onisiphorou et al. (2011) used 10 kN/m$^3$ as a mean value and 1 kN/m$^3$ and 3 kN/m$^3$ deviations from the mean in two different stability analysis scenarios. In the other study, Babu et al. (2012) collected the literature data until 2006, and estimated the range of unit weight average from 9.2 kN/m$^3$ to 13.1 kN/m$^3$, while standard deviation changes between 1.9 kN/m$^3$ and 3.4 kN/m$^3$.

2.4.2 Friction angle and Cohesion Initial Distributions (Literature-data)

The stability of the landfill depends on the shear strength parameters of MSW, as well. Similar to the soil, friction angle and cohesion are the parameters to evaluate shear strength of MSW. Associated uncertainties of MSW shear strength parameters are significant. MSW heterogeneity, sampling, and testing complications have been classified as obstacles of shear strength evaluation (Bray and Zekkos, 2009).

Referring to Appendix B, the friction angle and the cohesion data have been tabulated from related literature sources. These data include experimental research on direct shear (Table B-2), triaxial (Table B-3) and simple shear apparatuses (Table B-4), back analysis from failed landfill slopes (Table B-4), and suggested values by experts on this matter (Table B-5).

Figure 2-10: (a) Friction angle data histogram (b) Probability plot for normality
To assess the uncertainty of shear strength of MSW, this study relies on the set of measured data in Appendix B. Figure 2-10 illustrates the histogram and normal distribution for friction angle based on literature data (Appendix B). The friction angle uncertainty is expressed in terms of mean and standard deviation (Figure 2-10(a)).

Assessing the normality of this parameter, Kolmogorov-Smirnov and Shapiro-Wilk (at the 5% level of statistical significance), statistical methods did not show significant evidence of non-normality. Moreover, the relatively straight line on probability supports the hypothesis that these data are described by the normal distribution (see Figure 2-10(b)).

The reported friction angle values ranged from 0° to 53° (Appendix B). The normal distribution with 28.1° as a mean and 9.9° as a standard deviation can describe the prior distribution of friction angle. In comparison, Babu et al. (2012) obtained 32.3° for the mean angle and arrived at a standard deviation ranged between 6.1° and 9.7°. Onisiphorou et al. (2011) used 29° and 3° to 6° for the mean and the standard deviation respectively in order to estimate friction angle uncertainty.

In terms of cohesion data (Appendix B), 42 data out of 134 are lower than 5 kPa. The effect of human errors, apparatus calibration and different standards can potentially reduce the accuracy of different methods and reliability of small value test results. Hence, the cohesion values lower than 5 kPa have been assumed as censored data for this study. There are various methods for estimating parameters for censored data. Cohen’s test is the powerful way to estimate mean and standard deviation of censored normal distribution. Cohen’s method assumes that censored data are following the normal distribution same as observed data; however, they are below the detection limit. As the observed data follow the lognormal distribution (Geometric Mean=19.1 kPa, Standard Deviation=1.9 kPa), Cohen’s method adjusted cohesion geometric mean and standard deviation to account for data below the detection limit. Adjusted geometric mean and standard deviation are 10.4 kPa and 3 kPa, respectively. Figure 2-11(a) demonstrates detected cohesion data histogram.

Figure 2-11(b) illustrates the probability plot of observed values and demonstrates that lognormal distribution is an appropriate characterization of the data.
Three dumpsite failures (Sarajevo, Istanbul and Bandung) and three landfill failures (Ohio, Bogota and Durban) are described herein. The pre-found initial distributions of unit weight, friction angle, and cohesion of the waste materials utilized as uncertain inputs of the back-analysis. Although, pore water pressure conditions are not measured within the following six case studies, pore pressure ratio ($R_u$) considered based on hydrology regime and observed facts case by case.

**Sarajevo Dumpsite Failure (1977)**

The first identified MSW flow slide in the technical literature belongs to the Sarajevo dumpsite, 6 km away from the city borders. In December 1977, 200,000 m$^3$ of waste slid down traveled and progressed more than 1km further from the site. No compaction is reported to have been taken for this dumpsite. Blight (2008) pointed out that translational wedge failure took place in this dumpsite which is potentially due to the winter rain infiltration into the
Figure 2.12: Sensitivity range of unit weight (brown line), cohesion (blue line), friction angle (green line) and pore pressure ratio (red line) to factor of safety for (a) Sarajevo, (b) Istanbul, (c) Bandung, (d) Bogota, (e) Durban, (f) Ohio

Figure 2-12: Sensitivity range -of unit weight (brown line), cohesion (blue line), friction angle (green line) and pore pressure ratio (red line)- to factor of safety for (a) Sarajevo, (b) Istanbul, (c) Bandung, (d) Bogota, (e) Durban, (f) Ohio
uncompacted and uncovered waste. Figure 2-13 illustrated the failure zone of the landfill and its geometry.

Since, the high pore water pressure might have caused this failure, pore pressure ratio average for stability back analysis is considered equal to 0.3 with 0.1 deviation from the mean. Figure 2-12(a) shows the sensitivity of different parameters to the factor of safety for this slope analysis. The row vector in Zhang et al. (2011) equation is obtained with the slope of lines at intersection (correspond to the FOS coordination).

As shown in Figure 2-12, the sensitivity of different parameters to the factor of safety is approximately linear, close to the factor of safety with the mean values of input parameters \( g(\mu_0) \). The positive slopes detect the enhancing effects on FOS and vice versa.

The highest slope on Figure 2-12(a) belongs to the friction angle (green line). This, as well as the comparison between prior and posterior distributions, confirms that the friction angle is the most sensitive parameter to the factor of safety. The prior and the posterior probability density functions for the friction angle, cohesion, unit weight, and pore pressure ratio are compared in Figure 2-14.
Istanbul Dumpsite Failure (1995)

This dumpsite is about 30 km from Istanbul, Turkey. However, it is located on the upper side of the hill, which caused 39 people deaths from the waste movement on 11 informal brick-built houses. The waste materials were placed without any protection at the base, and there was no compaction and daily cover of the waste. Figure 2-15 illustrated the Istanbul failure.
Heavy rain and excessive infiltration are reported to be the major causes of the dump failure (Blight, 2008; Stark et al., 2009). Blight (2008) obtained $R_u = 0.14$ for this failure, while Stark et al. (2009) assumed this value equal to 0.41. Based on the previous research, the prior distribution of pore pressure ratio is assumed with the average of 0.25 and standard deviation of 0.1. Figure 2-16 shows the geometry of Istanbul landfill failure.
According to Figure 2-12(b), friction angle (green line) and cohesion (blue line) affect the factor of safety positively, while unit weight (brown line) and pore pressure ratio (red line) illustrate a negative effect.

![Graphs showing comparison of prior and back-analyzed distributions based on Istanbul dumpsite slope analysis](image)

*Figure 2-17: Comparison of prior and back-analyzed distributions based on Istanbul dumpsite slope analysis*

The back-analysis highly reduced the standard deviation of friction angle in comparison with other pre- and post-distributions. In this back-analysis, in order to increase prior factor of safety (FOS=0.869) to posterior factor of safety (FOS=1±0.05), pore pressure ratio distribution is skewed to the left and friction angle distribution is become leptokurtic (Figure 2-17).

**Bandung Dumpsite Failure (2005)**

One of the most recent landfill failures occurred in February 2005 in Bandung (Indonesia) which covered 200-250 m width and 900 m length of this city. Two and half million cubic meters of waste buried more than 147 people of local residents. These people were the informal
recyclers that live in shack homes around dumpsite. The thin layer of silty-clay separates the waste materials from bottom rock. Koelsch et al. (2005) believed that the slope failure took place within this silty-clay layer. Figure 2-18 illustrated the Bandung dumpsite failure.

![Figure 2-18: Bandung dumpsite failure (Blight, 2008)](image)

The failure zone of the Bandung dumpsite has been shown in Figure 2-19. In this study, the shear strength characteristics of silty-clay layer assumed same as Koelsch et al. (2005) including: 18 kN/m³ for unit weight, 10 kPa for cohesion and 12° for friction angle.

Koelsch et al. (2005) described Bandung dumpsite stability problems divided into two parts. On one side was the heavy nonstop rainfall for 3 days and, consequently, the water pressure increase in the waste and subsoil, and on the other side, was the severe damages due to the frequent fires reducing the reinforcement of the slope. Simplifying these trigger mechanisms, pore pressure ratio in waste is distributed with mean= 0.3 and standard deviation= 0.1. The pore pressure ratio for wet subsoil is assumed equal to 0.5 in this model.
Figure 2-12(c) shows the sensitivity of different waste parameters to the factor of safety. It is clear that unit weight contains the most effects on slope stability of Bandung dumpsite. The factor of safety based on the mean values of input parameters is equal to 1.02 which is lower than the model imperfection mean value plus one (FOS=1.02 < 1±0.05). So, there is no need to supply back-analysis and the pre and post distributions for input parameters remain the same for this stability simulation.

**Bogota Landfill Failure (1997)**

This bio-reactor landfill was located 20 km away from Bogota city in Colombia. The natural clay covered by PVC made up the liner of the landfill. A leachate collection system including pipes in a gravel bed, leachate pumping station and injection pipes for leachate recirculation were designed for this landfill.

The landfill failure started with toe collapse and continued with series behind it. Figure 2-20 demonstrates the failure zone and landfill geometry. The major reason of this failure was concluded to be the high pore water pressure due to the inappropriate leachate recirculation (Blight, 2008). The pore pressure ratio in waste and subsoil assumed 0.5 as a mean value to provide this trigger mechanism.
The sensitivity analyses are demonstrated on Figure 2-12(d) for Bogota site. The first factor of safety for this analyse obtained 1.157. To reduce this amount to unity, Figure 2-21 shows waste parameters changes in terms of normal or lognormal distributions. High unit weight value can be referred to waste compaction, while low friction angle and high pore pressure ratio are because of leachate recirculation.

Figure 2-20: Bogota Landfill failure cross section

Figure 2-21: Comparison of prior and back-analyzed distributions based on Bogota dumpsite slope analyse
Bogota landfill stability was highly dependent on unit weight of the waste materials in comparison with the other characteristics.

**Durban Landfill Failure (1997)**

The Bulbul landfill in Durban, South Africa, is a co-disposal landfill that was designed to be filled with specific “co-disposal ratio” of liquid and dry waste. To increase the slope stability, the berms have been provided across the toe of the landfill for each phase of disposing. Phase A has a compacted clay liner while, Phase B contains the combination of clay and geo-membrane liner.

![Figure 2.22: Durban Landfill failure cross section](image)

In September 1997, the landfill suddenly failed, and the waste moved on a prepared area for future phase of the landfill. Failure zone and landfill geometry are shown on Figure 2-22. The high pore water pressure in waste body is probably the main reason of failure due to (Blight, 2008):

1. Omitted drainage provisions of the design
2. Changed co-disposal ratio with greater volume of liquid with the same mass of dry waste

Considering the mentioned evidence, it is clear that berms and liners were provided the huge barrier around the saturated waste materials. To simulate this situation, the prior pore pressure ratio for waste assumed equal to 0.5.

Figure 2-23 illustrates the back-analysed unit weight, friction angle, cohesion, and pore pressure ratio.
Ohio Landfill Failure (1996)

In March 1996, the largest slope failure in the United States happened a few days after the 45 m extra excavation in front of the landfill toe (Stark and Eid, 2000). In addition, the site was overfilled by 13 to 15 m at the time of the failure. The translational failure is more probable because of the initial deep cracks at the top of the slope and the block form slide (Stark and Eid, 2000). Therefore, the slip surface possibly passed through saturated brown native soil (Figure 2-24). To simulate this situation, the geotechnical characteristics of subsoil are extracted from the research done by Stark and Eid (2000). It should be noted that the pore pressure ratio was assumed to be 0.3, similar to the previous research.
Using the prior distributions of waste materials as well as subsoil geotechnical characteristics, Ohio landfill failure back-analysed, and the potential posterior distributions compared with prior distributions in Figure 2-25.
2.5 Discussion

Sarajevo, Istanbul, and Bandung dumpsites as well as Ohio, Bogota, and Durban landfill failures were back-analysed in terms of slope stability. The distributions of Shear strength characteristics of the waste materials are improved in this process. The Figure 2-26 compared the posterior distributions of these parameters for each of these different sites.

In terms of the unit weight, all of the distributions followed almost similar mean values 10.6 kN/m$^3$ - 11.6 kN/m$^3$ and standard deviations 3.1 kN/m$^3$ - 3.7 kN/m$^3$, except Bogota case (Figure 2-26(a)). Since, Bogota bioreactor landfill failure happened due to the inappropriate leachate recirculation, and eventually higher moisture content rather than conventional landfills, the unit weight may be “routinely in excess of 15 kN/m$^3$” (Kavazanjian, 2001). In this study, the unit weight normal distribution for Bogota landfill obtained 14.8 kN/m$^3$ and 2.5 kN/m$^3$ as mean and standard deviation, respectively. The global distribution of unit weight is obtained with 11.1 kN/m$^3$ mean value and 3.6 kN/m$^3$ standard deviations, excluding Bogota landfill failure.

The cohesion lognormal distribution of waste materials did not experience major changes by these 6 back-analyses (Figure 2-26(b)). As a result, the cohesion obtained 10 kPa with 3 kPa deviation from the geometric mean value.

Based on the research done in this project, unlike cohesion, the friction angle varied in different case studies (Figure 2-26(c)). Except for the Istanbul dumpsite, the mean values of friction angle distributions were obtained lower than prior distribution (28.7°) by 1° to 6°. However, there is no significant evidence confirming that the friction angle distribution for Istanbul dumpsite failure is different from the other probabilistic distributions. Based on Central Limit Thorium (CLT), the approximate mean value for waste friction angle distribution for these case studies obtained 25.4° and the standard deviation of this normal distribution calculated 8°.

Figure 2-26(d) demonstrated the pore pressure ratio for all failure cases. The lowest pore pressure ratio is located in Istanbul site (mean=0.24), when compared with the other sites. The steep hillside (27°) of this dump may have caused leachate runoff from the tipping platform of the waste deposit area and provide less pore water pressure in comparison with other dumpsites and landfills. Blight (2008) obtained pore pressure ratio for Istanbul dumpsite equal to 0.14. High pore pressure ratio for landfills is demonstrated in case simulations.
Figure 2-26: Posterior distributions - of (a) Unit weight, (b) Cohesion, (c) Friction angle and (d) Pore Pressure ratio ($R_u$) - for back-analysed slopes
2.6 Conclusions

Landfill or dumpsite slope failure study is a new concept which has remained unexplored in recent years. However, most of the present landfill slope stability designs have relied on the factor of safety value, which is calculated based on constant values for municipal solid waste materials. Although it is common to use uniformly constant values for MSW properties in stability analyses, one should consider spatial and temporal variability and heterogeneous nature of MSW overburden pressure and decomposition. In comparison to deterministic stability analyses with constant input values, probabilistic approach can provide stability analyses considering MSW geotechnical uncertainties.

This research provided the probabilistic distributions of unit weight, friction angle, and cohesion based on literature data. Thus, the normal distribution of unit weight histogram founded with 10.9 kN/m$^3$ and 3.7 kN/m$^3$ as a mean and standard deviation values, respectively. In addition, the literature data conducted normal distribution with 28.1° as a mean and 9.9° as a standard deviation for friction angle uncertainty and lognormal distribution for cohesion with 10.4 kPa and 3 kPa as mean and standard deviation, respectively. These distributions have been considered as the prior distributions for slope stability back-analyses in this study.

In this study, three dumpsites and three landfill failures have been back-analysed in terms of slope stability. The distributions of unit weight, friction angle, and cohesion of the waste materials are improved in back-analyses. The improved distributions (posterior distributions) compared for the mentioned case studies and the optimized probabilistic distributions of these parameters were attained by the end of this chapter:

- **Unit weight**: Mean=11.1 kN/m$^3$
  Standard deviation=3.6 kN/m$^3$

- **Friction Angle**: Mean=25.4°
  Standard deviation=8°

- **Cohesion**: Geometric Mean=10 kPa
  Standard deviation=3 kPa
Chapter 3
Landfill Failure Mobility Analysis

3.1 Introduction

In 1977 a landfill in Sarajevo (Yugoslavia) failed, leading to a movement of 200,000 m$^3$ of waste to a distance of up to one kilometer. Despite huge asset and environmental damages, no deaths were reported in this failure. Since then, in addition to environmental devastation, landfill failures causing fatalities have occurred in various parts of the world. A catastrophic Payatas landslide in July 2000 completely covered a valley with 30,000 m$^3$ of waste and killed hundreds of people. The Bogota landfill in Columbia failed in 1997 and the waste travelled by 500 m, leading to creation of a waste dam on the river, which polluted soil and water. One of the most recent landfill failures occurred in February 2005 in Bandung (Indonesia), where the waste movement due to the failure encompassed an area of 200-250 m in width and 900 m in length. Around three million cubic meters of waste buried more than 147 people and destroyed rice fields (Koelsch et al., 2005).

The hazard associated with landfill slope failure may cause significant risk if there is a consequence arising from the failure. It is possible to experience a high probability of hazard and a low probability of risk due to the low vulnerability (a vulnerable element, e.g., human, property, soil, water sources, etc. could be located far away from landslide arising from slope failure). Thus, quantitative estimation of post-failure motion is vital, to estimate the extent of the endangered area and eventually the risk of landfill failure (Quan, 2012).
Generally, landslide is defined as “the movement of a mass of rocks, earth or debris down a slope” which can be either natural or a result of human activity (Rotaru, 2007). Landfills can be classified as man-made structures and their failures can be denoted based on human activity. Based on Rotaru (2007), three major factors which controls the potential of landfill failure has been identified as:

1. slope gradient: Sarajevo, Istanbul, Payatas and Bandung dumpsite failures, with approximately 45 degree slope prior to slope failure, suggesting that steeper slopes increase the failure likelihood.

2. Waste Compaction: Dumpsites have no systematic compaction (Koelsch et al., 2005). In principle, the absence of waste compaction reduces the rainfall surface flow and increases the rate of water percolation into the waste. Infiltrated water may result in creation of excessive pore water pressures and reduces the waste shear strength. Further, Blight (2008) determined that low waste compaction may lead to less cohesive waste material.

3. Water Pressure: Excessive pore water pressure reduces the waste shear strength. Hence, ten days of heavy rainfall in Payatas landfill and extra leachate injection in Bogota bioreactor landfill are examples where pore water pressure affected landfill slope failures (Bauer et al. 2008).

Rotaru et al. (2007) listed different movement types of landslide including: fall, topple, slide, spread, flow and complex. Considering the waste material type, trigger mechanisms of failure and reported landfill failure characteristics, “flow-type” may potentially describe landfill post failure movements. Differential shear strain along the slip surface is indicated in flow-type, as experienced in landfill failures. The Kettleman US landfill failure (1988) occurred along the base layer due to “insufficient strength of the geo-synthetic” (Adamczyk, 2005). In another study, Koerner and Snoog (2000) concluded that “wet clay beneath the geo-membrane” and “excessively wet foundation soil” is two liquid-related trigger mechanisms of landfill slope failures. Depending on the water content and failure movement velocity, waste-flow can resemble “debris-flow” which is one of the flow-type movements (there are different flow-type movements such as rock-flow, earth-flow, debris-flow and mud-flow). Rapid movement of the saturated materials during the failure is the main characteristic of this flow-type. Considering the
listed landfill failures (Table A-1), excessive water pressure due to the heavy rainfall and leachate build up saturates the waste materials. In addition, fatalities resulting from landfill failures may arise due to rapid movement of a waste preventing people from evacuation.

Since the early works of Hungr (1995), landslide researchers have tried to better understand landslide mechanisms and to predict flow rheology. The rheology of the flow is denoted as “the resistance forces interact inside the flow and at the interface between the flow and the bed path” (Quan, 2012). Generally, rheology models are divided into cohesive and frictional types. Debris-flows as a mixture of solids and fluids are categorised as a frictional type rather than a cohesive type of mud-flows. The simplified frictional model and Voellmy’s model are the two frictional types of rheology. Voellmy’s model considers the effects of flow turbulence in addition to basic frictional feature. Koerner (1976) reported overestimation with frictional rheology while the Voellmy model provides a much better estimation. Recently, the Voellmy model is founded in agreement with observed global flow behaviour of landslides (Quan, 2012). Researchers (Hungr, 1995; Revellino et al., 2004; McDougall and Hungr, 2005; Quan, 2007; Hurlimann et al., 2008) extensively used this common type of rheology in dynamic models and came up with reasonable results. In this study the waste-flow is assumed to follow the Voellmy rheology.

Voellmy (1955) defined the resistance force (SF) as:

$$SF = f + \frac{u^2}{\xi h}$$

[3-1]

where $f$ is the friction coefficient, $u$ is the flow velocity (m/s), $\xi$ is the turbulence coefficient (m/s²) and $h$ is the flow depth (m).

$$f = \tan(\varphi_b) = (1 - r_u) \times \tan(\varphi),$$

[3-2]

where $\varphi_b$ is bulk basal friction angle, $r_u$ is pore water pressure ratio and $\varphi$ is dynamic basal friction angle.

Unlike the small landslide models based on Coulomb’s Law (Kinematics of sliding), the physical behaviour of excessive travel distance of the observed catastrophic landslides is hard to
predict. This fact may depend on different reduction mechanisms of basal friction. Quan (2012) explained reduction mechanisms within flow path and flow material intersection:

1. Cushions of trapped air due to the water vapor by the friction heat can lubricate the flow. Specifically, gas generation in landfills by various kinds of biochemical and chemical reactions may increase trapped air.

2. Originally smooth bed materials such as limestone, gypsum or glaciers as well as the regular bed materials which are made smooth by frictional melting may lubricate the flow.

3. The saturation of the flow materials increases the liquid content of the flow and may reduce the interface friction between bed materials and flow materials.

However, in addition to the above reduction mechanisms of basal friction, the dynamic basal friction ($\varphi$) is normally less than static basal friction. Considering the effect of pore water pressure ratio ($r_u$), the bulk basal friction angle ($\varphi_b$), would range between $3^\circ$ to $11^\circ$.

Recently, several numerical models such as MADFLOW (Chen and Lee, 2007), TOCHNOG (Crosta et al., 2003), RAMMS (Christen et al., 2010), DAN3D (Hungr and McDougall, 2009) and DAN-W (Hungr, 2010), have been developed to simulate the run-out of landslides. These numerical models are able to compute different run-out variables including travel distance (run-out distance), thickness, and velocity. The computed outputs can be associated with vulnerability for a quantitative risk assessment (Quan, 2012).

Sudden slope failures of landfills as man-made structures continue to claim lives and destroy properties as well as pollute the environment. Waste travel distance after the landfill failure is critically important to calculate the extent of endangered areas which can be developed by numerical models.

Dynamic run-out models are commonly used for back-analysis of past events rather than predictive modeling. These models are sensitive to friction parameters, which leads to the lack of reliable calibration. This is the basic limitation of run-out models (Quan, 2012). However, recent investigations contain a number of back analyses to calibrate input parameters for run-out models in terms of rock, debris and soil material failures (Hungr and Evans, 1996; Revellino et
al., 2004; McKinnon, 2010). But as per the extensive literature review conducted as part of this research, no literature was identified to calibrate run-out models for waste materials.

To summarize, some of the work on run-out model calibrations: Hungr and Evans (1996) back-analysed 23 well-recorded rock avalanches with DAN-W software. Simple Frictional, Voellmy and Bingham rheologies have been alternatively used for all events. Both simple frictional and Bingham rheologies overestimated the landslide velocities while the Voellmy rheology obtained a good fit. The best estimation of travel distance and thickness is also obtained by the Voellmy rheology (Hungr and Evans, 1996). Revellino et al. (2004) presented successful calibration results, which were obtained after 19 back-analyses of similar debris avalanches. A single set of the Voellmy rheology input parameters was employed in Revellino et al. (2004) calibration. Using statistical approach, McKinnon (2010) examined frictional and Voellmy rheologies by DAN-W Software to investigate run-out models of 40 rapid flow-like landslides. Normalized mean values and associated standard deviations for run-out parameters were provided in the McKinnon (2010) study and recommended as a reliable range for predictive modeling of future events.

Although the technical literature mostly contains constant values as resistance parameters (e.g., friction and turbulence coefficients for Voellmy rheology calibration to predict future run-out events), McKinnon (2010) emphasized that the calibrated resistance parameters should not be deterministic. This is because the failure can happen via various pathway situations of run-out, and the characteristics of flow materials are potentially different spatially and temporally. Thus, it is not logical to claim constant values as resistance parameters, while a “realistic range of parameters” may provide a probable condition for landslide run-out prediction (McKinnon, 2010). Given this, the objective of this study is to provide an analysis tool (methodology) for modelling of waste-flow in the event of landfill failure, approached by calibrated probabilistic distributions of resistance parameters. These parameters are obtained by performing the back-analyses of previous landfill failures.
3.2 Methodology and Material

The dynamic software model, DAN-W, developed by Hungr (1995), has been used in this study to back-analyse the extent of waste movement from the landfill failures. This is the first time that DAN-W software has been applied to model waste material flow after a landfill failure. So, similar to the most common debris-flow models in the literature (Hungr, 1995, Hungr and Evans, 2004; Pastor et al., 2007; Mckinnon, 2008; etc.), Voellmy rheology has been chosen as the rheology kernel to simulate waste-flow. In terms of the calibration, this rheology contains three major factors including frictions and turbulence parameters. The friction factors (friction coefficient and initial friction angle) mostly effects the travel distance of the flow, while the turbulence factor frequently influences the velocity of the flow (Mckinnon, 2010). High ranges of turbulence coefficient potentially reduce the effect of velocity on resistance force (SF).

Although DAN-W software can incorporate variable characteristics, including point to point displacements, thicknesses, velocities and so on, the focus of this study is on the travel distance of waste-flow, following landfill slope failure. Because the maximum run-out distance is an apparent feature of the flow which can be measured with high accuracy in any time after the failure, temporal characteristics such as velocity, is required on-time monitoring or indirect calculations which are not applicable and reliable for the landfill failure case studies.

Three dumpsite failures (Sarajevo, 1977; Istanbul, 1995 and Bandung, 2005) and two landfill failures (Ohio, 1995 and Durban, 1997) have been investigated in this study. Considering the fact that these slopes have already failed, the travel distances of the waste movement after the landfill failures have been used to back-analyse the run-out models and improve estimates of primary parameters. The calibrated input parameters are obtained in back-analysis loops when the travel distance of the run-out model equals to the observed travel distance.

Voellmy rheology contains fixed and variable factors. Here, it is assumed that the friction coefficient and initial friction angle are variable factors. In the second chapter, the normal distribution of friction angle of waste materials is resulted with 25.4 for mean value and 8 for standard deviation. So, in this study the range of friction coefficient factor has been calibrated as a probabilistic distribution.
Due to the lack of information about waste-flow velocity after the landfill failures, as per the McKinnon (2010) recommendation, the turbulence coefficient is assumed equal to 1500 m/s² in all of the landfill failure models. In addition, the model’s predicted velocities have been checked and all were seen to exceed the human running speed (5 m/s). As a result, all of the landfill failure case studies have been described as catastrophic or fatal events. Table 3-1 illustrates fixed characteristics of the waste materials in Voellmy rheology and default DAN-W parameter values which have been used in the back-analyses.

Table 3-1: Default DAN-W parameter values used in back-analyses (Hungr, 2010)

<table>
<thead>
<tr>
<th>Control Parameters</th>
<th>Default values</th>
<th>Material Parameters</th>
<th>Default values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of elements</td>
<td>50</td>
<td>Unit weight</td>
<td>11.1 kN/m³</td>
</tr>
<tr>
<td>Time intervals</td>
<td>0.02-0.1 s</td>
<td>Turbulence Coefficient</td>
<td>1500 m/s²</td>
</tr>
<tr>
<td>Smoothing coefficient</td>
<td>0.02</td>
<td>Erosion Depth</td>
<td>0.0</td>
</tr>
<tr>
<td>Tip ratio</td>
<td>0.5</td>
<td>Internal Friction Angle</td>
<td>25.4°mean &amp;</td>
</tr>
<tr>
<td>Stiffness coefficient</td>
<td>0.05</td>
<td></td>
<td>8° standard</td>
</tr>
<tr>
<td>Stiffness ratio</td>
<td>5</td>
<td></td>
<td>deviation</td>
</tr>
<tr>
<td>Centrifugal force</td>
<td>On</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boundary block geometry</td>
<td>Vertical</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pressure term</td>
<td>modified</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To estimate the primary distribution of friction coefficient for back-analyses, this study reviewed relevant studies on landslides and introduced statistical distributions for this parameter. McKinnon and Hungr (2008) estimated typical friction coefficient ranges of 0.07 and 0.2 for debris-flow and 0.03 to 0.24 for rock avalanche models with DAN-W software. In the other study, McKinnon (2010) estimated a 0.1 for friction coefficient in flow-type movement. However, Quan (2012) indicated 0.162 and 0.136 for the mean and standard deviation, respectively, of the best-fit normal distribution to the friction coefficient histogram of 168 studied landslides. In this study, the primary distribution of friction coefficient assumed normal distribution with 0.15 for mean and 0.1 for standard deviation. This primary distribution has been used as an input in the back analyses.

Tarantola (2005) stated the simplified probabilistic back-analysis equation for normally distributed parameters. Assuming \( \mu_\theta \) as a mean and \( C_\theta \) as a covariance matrix of \( \theta \) primary...
distribution, the objective of probabilistic back-analysis is then to improve the probabilistic distribution of $\theta$ to $\mu(\theta|d)$ and $C(\theta|d)$:

$$\mu(\theta|d) = \mu_\theta + C_\theta H^T (HC_\theta H^T + \sigma_\epsilon^2)^{-1} [1 - g(\mu_\theta) - \mu_\epsilon]$$  \[3-3\]

$$C(\theta|d) = \left( \frac{H^T H}{\sigma_\epsilon^2} + C_\theta^{-1} \right)^{-1}$$  \[3-4\]

$$H = \frac{\partial g(\theta)}{\partial \theta} \bigg|_{\theta=\mu_\theta}$$  \[3-5\]

Defining $g(\mu_\theta)$ as a ratio of travel distance in the run-out model over observed travel distance in reality, when this factor was equal to unity, then the back-analysis loop is completed. “$H$” is measured slope representing the sensitivity of $g(\theta)$ with respect to $\theta$ at point $\mu_\theta$. Quantifying the effect of model imperfection, $\mu_\epsilon$ and $\sigma_\epsilon$ are considered as the mean and standard deviation of this factor. Regarding uncertainties of the simulation, $\mu_\epsilon = 0.01$ and $\sigma_\epsilon = 0.01$ are suggested.

The limitation of this method is that it is applicable only when $g(\theta)$ is largely linear around point $\mu_\theta$, which has been satisfied in this study.

Figure 3-1 illustrates the flowchart of the methodology. Using literature data to characterize the friction prior distribution, two landfill failures and three dumpsite failures were back-analysed to estimate the friction posterior distributions. Finally, the optimized probabilistic distributions for friction coefficient have been provided. These results can be used for project-specific design when data are not available. Having the friction coefficient distribution, the range of travel distance can be obtained with Taylor’s series method (Appendix C).
3.3 Results

3.3.1 Sarajevo Dumpsite Failure (1977)

The MSW flow slide reported in the technical literature for Sarajevo dumpsite, located 6 km away from city borders. In December 1977, 200,000 m$^3$ of waste slid down and traveled 900 m. No compaction is reported for during the operation of this dumpsite. Although there were no injuries or deaths, two bridges and five houses were destroyed and extensive contamination of the environment was reported as two stream beds got filled with waste materials.

Figure 3-2 illustrates the Sarajevo dumpsite simulation after the failure with DAN-W. The best-fit travel distance obtained when friction coefficient is equal to 0.148. The thickness of the waste deposit after 850 m run-out from site location gained 2.5 m which is equal to Blight’s (2008) estimation for the same distance (1.5-2.5 m).
3.3.2 Istanbul Dumpsite Failure (1993)

This dumpsite is about 30 km away from Istanbul, Turkey. However, it is located in the upper side of a hill, which caused 39 deaths due to the running in of the waste on failure, with 11 demolished informal brick-built houses.
Due to a sewer fracture and dammed waste in front of the sewage, serious environmental damage occurred. The waste materials were placed without any leakage protection (e.g., compacted clay or geo-synthetic) and there was no compaction and daily cover of the waste on the dumpsite.

The maximum travel distance of waste-flow obtained 170 m while the mean value of friction coefficient distribution was equal to 0.135 in Voellmy rheology (see Figure 3-3). The waste depth on the bottom of valley reached 17.5 m which is close enough to Blight (2008) estimation (16 m).

3.3.3 Bandung Dumpsite Failure (2005)

One of the most recent dumpsite failures occurred in February 2005 in Bandung (Indonesia) which covered 200-250 m width and 900 m in length. Two and one-half million cubic meters of waste buried more than 147 people. These people were informal recyclers that lived in shack homes around the dumpsite. The top side of the landfill collapsed on the residential area. The left side of the landfill, which was surrounded by rice fields, was then covered by the waste. The amount of waste from the landfill developed 4-5 times more than its original placement volume. This man-made disaster resulted in huge impacts to the environment and human life.

Figure 3-4: Pre and Post failure profile of Bandung dumpsite

Figure 3-4 demonstrates the run-out modeling of Bandung failure in DAN-W. The maximum waste-flow simulated (1 km) in a condition with 0.122 for friction coefficient in
Voellmy rheology. Although, Blight (2008) claimed that waste thickness was equal to 3-4 m in most of the flow path, this depth happened in the last 200 m of the run-out simulation.

3.3.4 Durban Landfill Failure (1997)

The Bulbul landfill in Durban, South Africa, is a co-disposal landfill which was designed to fill with specific “co-disposal ratio” of liquid and dry waste. To increase the slope stability, surrounding berms were provided across the toe of landfill for each phase of disposal. Phase A had a compacted clay liner while Phase B contained the combination of clay and geo-membrane liner.

In September 1997, the landfill suddenly failed and 150-180 thousand cubic meters of the waste moved on the prepared area for a future phase of the landfill. Because the failure happened in the waste disposal area, no deaths or injuries occurred and the environmental destruction was limited.

As can be seen, based on Figure 3-5, the waste-flow started from 346 m on the horizontal axis and continued about 80 m. The maximum travel distance is obtained when Voellmy frictional parameter mean value was equal to 0.173. The depth of the flow on the toe location after the failure gained 15 m in DAN-W simulation while the Blight et al. investigative team (2004) estimated 12.5 m for waste thickness in that location.

Figure 3-5: Pre and Post failure profile of Durban landfill
3.3.5 Ohio Landfill Failure (1997)

On March 1996, the largest slope failure in the U.S. happened a few days after a 45 m excavation in front of the landfill toe. In addition, the site was overfilled by 13 to 15 m at the time of failure. Translational failure is more probable because of initial deep cracks at the top of the slope and block form slide (Stark and Eid, 2000). Hence, the failure surface potentially crossed saturated brown native soil.

DAN-W simulation for Ohio landfill is illustrated in Figure 3-6. The back-analysis mean value for friction coefficient in the Ohio landfill simulation is 0.156 when the travel distance of the simulation gained equals the real failure run-out. Stark and Eid (2000) investigated the waste thickness with four borehole. These boreholes were located on 270 m, 330 m, 350 m and 500 m on the horizontal axis (Figure 3-6). The waste depths in boreholes are obtained as 40, 25, 22, 18, respectively, while the simulated run-out came with 30, 30, 18 and 19 respectively, indicating a good compatibility.

![Figure 3-6: Pre and Post failure profile of Ohio landfill](image)

Figure 3-6: Pre and Post failure profile of Ohio landfill
3.4 Discussion

Sarajevo, Istanbul and Bandung dumpsite failures, as well as Ohio and Durban landfill slides have been back-analysed in terms of waste mobility. The distributions of the friction characteristics of the waste materials improved in this process. Figure 3-7 compares the posterior distributions of friction coefficient for different sites.

![Figure 3-7: The posterior distributions of friction coefficient for different sites](image)

Waste density and waste types are two of the most important parameters influencing the waste shear strength (Koelsch, 2007; Stark, 2000); hence, comparing with the friction coefficient of Voellmy rheology in this approach is of interest. As seen in Figure 3-7, higher friction coefficient distributions are associated with failed waste of landfills as opposed to dumpsites. Waste compaction on landfills may increase frictional texture in Durban or Ohio landfills and the berm structures on the toe of the Durban landfill possibly prevented failure movement and increased friction parameter in this case to the highest probabilistic distribution.

Furthermore, in the absence of high strength waste materials such as wood, metal, plastic, paper and cardboard, the friction can be destructively affected. Davis and Freeman (2000) reported 37% paper and cardboard, plastic, metal and glasses in combination with 31% food waste in the Durban landfill, while the Bandung dumpsite contained 15% high strength waste
and 82% food waste (Blight, 2008). The scavenging of the waste materials in Bandung dumpsite substantially changed the waste composition and reduced the shear strength, and eventually, the friction characteristics. This fact can potentially confirm lowest friction coefficient distribution for Bandung site.

Istanbul waste composition contained 21% frictional waste materials and more than 72% food waste as a result of the scavenging (Blight, 2008). These findings indicate the low mean value for friction coefficient distribution of Istanbul dumpsite, similar to the Bandung case study.

Although the Sarajevo dumpsite contained a high range of frictional waste materials (Blight, 2008), the uncompacted waste justifies the lower mean value for friction coefficient distribution for this dumpsite in comparison with Durban and Ohio landfills.

Based on the waste compaction and waste type effects, Table 3-2 lists the classification of the friction coefficient for different situations. The global distribution for friction coefficient calculated for each class is based on the back-analysis results.

<table>
<thead>
<tr>
<th>Class No.</th>
<th>Waste Compaction</th>
<th>Waste type</th>
<th>Global Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>Class I</td>
<td>No</td>
<td>Low strength</td>
<td>0.12</td>
</tr>
<tr>
<td>Class II</td>
<td>No</td>
<td>High strength</td>
<td>0.14</td>
</tr>
<tr>
<td>Class III</td>
<td>Yes</td>
<td>Low Strength</td>
<td>0.16</td>
</tr>
<tr>
<td>Class IV</td>
<td>Yes</td>
<td>High Strength</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Lower mean values for non-compacted and low frictional materials, as well as higher mean values for compacted and high frictional materials are classified in Table 2. As can be seen, the dumpsites, which are contained the non-compacted disposals, with low frictional materials (e.g., paper, glass, metal, etc.) belong in class I (e.g., Bandung dumpsite). The class II contains dumpsites with higher range of frictional materials (e.g., Istanbul and Sarajevo dumpsites). The class III and Class IV of global distribution for frictional coefficient is appropriate to use for landfills which are developed with proper compaction for waste layers, with low strength (e.g., Ohio landfill) and high strength (e.g., Durban landfill) waste components, respectively.
3.5 Conclusions

Waste travel distance after a landfill slope failure is crucially important for calculating the extent of endangered area and potential vulnerability of the elements at risk. Using appropriate rheology for the waste-flow, dynamic run-out models are essential tools to estimate waste movement after the failure. It is a first time that the dynamic software model, DAN-W, has been used to back-analyse the extent of waste movement after the landfill failures. Voellmy rheology has been chosen from rheology kernel to simulate waste-flow. Modeling three dumpsites and two landfills, the range of friction coefficient as the most sensitive parameter of Voellmy rheology given probabilistic distributions for each case study. Comparing the results, it has been concluded that waste compaction and waste type can potentially control the friction coefficient probabilistic distribution. Waste compaction on landfills may increase the friction coefficient in comparison with dumpsites. In addition, in the absence of high strength waste materials such as wood, metal, plastic, paper and cardboard, the friction is destructively reduced. Based on the waste compaction and waste type effects, four classifications of the friction coefficient were defined in this study.

For the first two uncompacted classes, the mean values of the friction coefficient distribution are, 0.12 and 0.14, when the waste types are low and high strength, respectively. Furthermore, for the class 3 with compacted and low strength waste materials, the mean is 0.16, and the class 4 devoted to high strength and compacted waste materials with 0.18 mean value. It is notable that, all of the friction coefficient distributions have the same standard deviations (SD=0.02).

These results can be used for project-specific design when data are not available. Having the friction coefficient distribution, the range of travel distance can be obtained with Taylor’s series method (Appendix C).
Chapter 4
Landfill Failure Risk Assessment Framework

4.1 Introduction

Stability of the refuse became a major concern in 1977 when a landfill in Sarajevo (Yugoslavia) failed which caused the translational movement of 200,000 m$^3$ of waste. The Bogota landfill in Columbia also suffered slope failure in 1997 and river was dammed by the waste and polluted soil and water. In the following years, apart from environmental devastation, landfill failures involving human fatalities occurred including the Payatas landslide in July 2000, which completely covered the valley by 30,000 m$^2$ waste and killed hundreds of people. One of the most recent landfill failures occurred in February 2005 in Bandung (Indonesia), which covered 200-250 m width and 900m length of Bandung city. In this landfill failure, 2.7 million cubic meters of waste buried more than 147 people and destroyed rice fields (Koelsch, 2007).

The risk analysis of the slope failure danger in refuse stability category is the objective of this study. There are risk literatures about pollution beyond geological barrier and gaseous emission, or general cost and environmental protection. Despite the dangerous nature and the catastrophic consequences, the literature review of landfill slope stability reveals that the waste landslide risk has not been assessed. This study reviewed this concept considering the previous research done on dam or natural mountain slope stability.

Risk is the measurement of adverse effect probability to life, health, property, or environment (Lacasse et al., 2007). Qualitative, semi-quantitative and quantitative evaluates are
three ways to analyse the risk (Heiniman, 1999). The suitability of the qualitative or quantitative risk evaluation depends on the range of the desired accuracy, problem nature and data availability (Dai et al., 2002). Quantitative risk analysis is characterized as being easy for communication and comparison to support management policies (Michael-Leiba et al., 2000; Bell and Glade, 2004). Yet, qualitative risk analysis has been suggested for small scale circumstances for which quantitative system is not economical (Fell, 1994).

Risk is the product of “magnitude-frequency” features of hazard as well as spatial and temporal vulnerability. The hazard of landfill failure can be risky only if there is a consequence for this failure. It is possible to experience a high probability of hazard and a low probability of risk due to the low vulnerability (the element is located far away from landslide). Thus, quantitative estimation of the post-failure motion is vital to know the extent of the endangered area and eventually the risk of landfill failure (Quan, 2012).

There are two well-defined quantitative risk values:

Firstly, special risk ($R_S$) is a useful definition for the individual element (e.g. one person life). This value is quantified by this equation (Fell, 1994; Lee and Jones, 2004):

$$R_S = H \cdot V \quad [4-1]$$

Secondly, the total risk ($R_T$) has been defined as a hazard of the event times the consequence (Lacasse et al., 2007; Chi et al., 2011). This can be written as:

$$R_T = H \cdot C = H \cdot V \cdot E \quad [4-2]$$

Where $H$ = hazard,

$V$ = vulnerability of elements at risk,

$E$ = the amount of the total loss of elements at risk (number of dead people, cost of demolished properties),

$C$ = consequence
Although the state of art for landslide risk assessment followed the similar attitude in literature, it is clear that the research methodologies are different. Regarding the problem nature, the available data, and uncertainties, there are various methods to evaluate hazard and consequence aspects of risk analysis. These categories are studied and different approaches are reviewed in this chapter.

4.1.1 Hazard Assessment

Traditionally, the natural and engineered slope stabilities have been assessed by the safety factor. This can be classified as a deterministic approach. Factor of safety is the function of conservative amounts for geotechnical characteristics of the slope materials. Unit weight, cohesion, and friction angle as geotechnical parameters contain uncertainties. Choosing the reasonable values for these parameters and determining the factor of safety fail to address suitable incorporation of uncertainties. The possible solution for this problem is hazard and risk approach (Lacasse, 2007). Hazard approach contains probability or reliability analysis as a complement to the deterministic approach (Duncan, 2000). Probabilistic analysis uses the means of evaluating uncertainties, and provides the mean of combined effects for factor of safety. In addition, the deviation of uncertainties is influencing the factor of safety distribution. However, the result of the hazard approach is the probabilistic distribution of factor of safety which contains the effect of uncertainties on mean and standard deviation values.

Figure 4-1, shows an example of deterministic approach inefficiency in comparison with probabilistic approach for slope stability analyses.

\[ \sigma(FS) \] is the mean value and \[ \mu(FS) \] is standard deviation for normal distribution. \[ \beta \] is reliability index, which is defined as:

\[ \beta = \frac{\sigma(FS)^{-1}}{\mu(FS)} \]
Assuming normal distribution for safety factor with detected mean and standard deviation, the probability of failure is defined as a probability that the factor of safety would be less than 1. Although, $\sigma(FS) = 2$ has obtained for distribution A, the probability of $FOS < 1$ is more than distribution B with lower factor of safety ($FOS=1.55$). So, the amount of reliability index ($\beta$) plays a certain role in these sorts of analyses.

There are several ways to assess the probability distribution of the factor of safety. Duncan (2000) suggested the Taylor series method to evaluate the factor of safety mean and the standard deviation in geotechnical projects such as slope stability, retaining wall stability or consolidation settlement. So, assuming mean and standard deviation for probabilistic distributions of input parameters ($IP_1, IP_2, \ldots, IP_n$) the output (OP) mean value can be obtained considering the mean values for all input parameters. To evaluate the standard deviation for output, following equation has been suggested for Taylor’s series method:

$$SD = \sqrt{\left(\frac{\Delta OP_1}{2}\right)^2 + \left(\frac{\Delta OP_2}{2}\right)^2 + \cdots + \left(\frac{\Delta OP_n}{2}\right)^2}.$$  \hspace{1cm} [4-4]

where $\Delta OP_1$ can be provided by increased and decreased $IP_1$ mean value by one standard deviation. And $\Delta OP_n$ can be provided by the increased and decreased $IP_n$ mean value by one standard deviation.
The other regular method for reliability analysis is first order reliability method (FORM) which has been developed by Hasofer and Lind (1974). The reliability index ($\beta$) for normally distributed parameters as an input is defined as:

$$\beta = \min_{x \in F} \sqrt{\left[ \frac{x_i - \mu_i}{\sigma_i} \right]^T R^{-1} \left[ \frac{x_i - \mu_i}{\sigma_i} \right]}$$  \[4-5\]

where $X_i$ is a random variable; $\mu_i$ and $\sigma_i$ are the mean and standard deviation of $X_i$, respectively; $R$ is correlation matrix of $X_i$; $F$ is a failure region (corresponding to the domain in which FOS $\leq 1$).

Using this method, Chi et al. (2011) analysed the hazard of the slope failure in central Taiwan. In this case study, the sensitivity analysis is utilized to investigate the effect of various degrees of parameter uncertainty. Based on the test results, shear strength parameters are normally distributed with the means and the coefficient of variation. Using FORM, sensitivity analysis provided the range of factor of safety (FOS), reliability index, and probability of failure.

Monte Carlo, one of the most popular methods to follow probabilistic approach, was developed initially by Metropolis and Ulam (1949). This method was first applied for hazard assessment of landslides by Calvo and Savi (2008).

To assess the hazard of slope, the input data includes the slope geometry, probable failure surface, shear strength, unit weight characteristics of the slope components, and trigger mechanism features such as rainfall intensity, water level, earthquake magnitude. Assigning a distribution for uncertain input parameters, Monte Carlo probabilistic method can be used. Each time, a randomly-picked value from probability distribution of input parameters is analysed, and the output value is calculated. This process has to be repeated many times to make a probabilistic distribution for output. The output of slope hazard assessment is the probabilistic distribution of safety factor.
4.1.2 Consequence Assessment (Vulnerability Assessment)

Vulnerability is the range of potential damage or loss of an element at risk (damage for property and loss for human) subjected to the hazard (Fell and Harford, 1997). This range is from 0 to 1 depending on the hazard and the interaction of elements.

Predicting the quantitative value for vulnerability, such as potential damage or degree of loss, is the hard task to achieve because “landslides are spatially discrete phenomena”. Unlike spatially continuous events such as earthquake, flood, and hurricane, it is hard to identify intensity measure for landslides (Laccasse et al., 2007; Lee and Jones, 2004). Based on literature, there are semi-quantitative and quantitative indices for vulnerability (Moon et al., 1992; Fell and Hartford, 1997; Li et al., 2010).

Recently, most of the research focused on quantitative vulnerability assessment (Kayna et al., 2008; Galli and Guzatti, 2007; Haugen and Kaynia, 2008; Quan, 2012). For example, Quan (2012) described the state of art to provide three physical vulnerability curves that relate quantitative inputs in specific points including height, velocity, and impact pressure of landslide, to economic loss were derived from well-documented debris-flow event. The height, velocity, and impact pressures in considered point are the landslide post-failure characteristics which are determined by the dynamic run-out models in Quan (2012) study.

Dynamic run-out models are able to evaluate the propagation of landslide materials after the failure. As a result, the suffered elements at risk can be obtained in the propagation zone. Therefore, the run-out models can potentially provide the appropriate inputs for the vulnerability evaluation.

However, Quan (2012) used dynamic run-out models as a tool to evaluate the hazard and vulnerability both, and eventually the quantitative risk. In contrast, Costra et al. (2005) coupled the stability and mobility modeling to separate the effect of risk components. Stability analysis was modeled using 2D Slope/W numerical code in order to build the hazard scenarios. The run-out was simulated using the quasi-three-dimensional finite element method to digitalize post-failure topographies of landslide site.
All in all, the lack of information in terms of quantitative vulnerability on landslides is undisputable. This fact is more tangible for landfill failures. Historical fatal landfill failures have shown that developing countries are facing huge challenges in terms of life loss risk. The landfill failure can potentially threat the lives of the landfill workers and local residents.

4.1.3 Risk Assessment

The final step of the risk analysis belongs to the risk assessment which carries out the result, and evaluates against judgement criteria (Fell and Ho, 2005). Risk comparison with the socially acceptable risk is the most popular judgment criteria in the risk analysis literature (Whitman, 1984; Reid, 1989; Fell, 1994). There are some critical aspects that affect this method:

1. Voluntary vs. Involuntary risk: There is enough evidence that the voluntary risk is more acceptable rather than the involuntary risk. In terms of landslides, natural landslides are a case of “voluntary risk” which is acceptable with $10^{-3}$ annual specific risk value (Starr, 1976; Reid, 1989). On the other hand, man-made structures, including landfills, are a case of involuntary risk which is acceptable with much lower annual risk value ($10^{-5}$), because the awareness of the risk problem will increase the expectation of the society (Fell, 1994).

2. Multiple-death vs. few number of death events: It is clear enough that society can accept higher risk for a smaller loss rather than a bigger one (Whitman, 1984). Whitman (1984) stated that the acceptable annual risk with 100 death people is 20 times less than the annual risk for one dead body (Figure 4-2).

Considering all mentioned points, the most difficult section of the risk assessment is the collection of acceptable criteria. The previous effort contains “Frequency-Fatality” (Fell, 1993; GEO, 1998) curves which have been used for different events to guide the risk levels and associated consequences which are acceptable by the society.
4.2 Objective

Landfill failure as a hazard, and life loss as major consequence of this event are constituted as the basis for the risk analysis in this research. Although the risk literature contains various research on natural landslide or dam failure as a man-made structure, landfill slope stability requires more research in this category. Landfill failure has to be investigated by risk analysis approach, because:

1. Landfill geotechnical characteristics contain a wide range of uncertainty due to the heterogeneity of waste disposal materials. Deterministic approach fails to deal consistently with uncertainties. Risk approach as a compliment with deterministic approach can compensate this imperfection.

2. Landfill is an engineered slope and should be classified as an involuntary risk. Society tolerates less risk while it is man-made structure.
3. However, catastrophic landfill and dumpsite failures have proven that considering merely the lowest factor of safety (only hazard) is not an encompassing criterion for designing, and the probable vulnerability as a result of failure may have to be considered in the design. This study includes the hazard and vulnerability in designing the landfill slope stability.

4.3 Methodology

In order to analyse the risk of landfill slope failure, based on the risk definition, two factors has been quantified: hazard and vulnerability.

In terms of the landslides, hazard assessment usually involves the combination of the probability analyses of specific slope and the frequency of slope failure occurrence (Fell and Ho, 2005). In terms of the landfill slope failure, the required data for probabilistic analysis of the slope failure involves landfill geometry and waste geotechnical characteristics. Chapter 2 provided the required data for waste unit weight, cohesion, and friction angle probabilistic distributions (Table 4-1).

<table>
<thead>
<tr>
<th>Waste characteristics</th>
<th>Distribution Type</th>
<th>Distribution</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight</td>
<td>Normal</td>
<td>11.1 kN/m³</td>
<td>3.6 kN/m³</td>
<td></td>
</tr>
<tr>
<td>Friction Angle</td>
<td>Normal</td>
<td>25.4°</td>
<td>8°</td>
<td></td>
</tr>
<tr>
<td>Cohesion</td>
<td>Lognormal</td>
<td>10 kPa</td>
<td>3 kPa</td>
<td></td>
</tr>
</tbody>
</table>

These data have been realized by back-analyses on three dumpsite failures and three landfill failures. To assess the hazard of the slope failure for specific cross section of the landfill, these uncertain distributions may be used as the input data for the slope stability analysis. Using Monte-Carlo method, the probability distribution of factor of safety can be computed. The slope failure can happen only if the factor of safety was lower than unity which is the probability of failure (P(FOS<1)).

IUGS (1997) predicted several methods to calculate the frequency of landslide occurrence including historic data, empirical methods, expert judgments, and the relationship to the
frequency of trigger mechanisms. Relating the frequency of trigger mechanisms, such as heavy rainfall, to the frequency of slope failure at a certain time is more practical in this study. Yet, heavy rainfall is the major cause of the landfill or dumpsite failures. Ten days of heavy rainfall in Payatas dumpsite failure (2000), and three days of heavy non-stop rainfall before Bandung dumpsite failure (2005) are the two catastrophic examples of rainfall as a trigger mechanism of landfill failure. In this study, considering the heavy rainfall as a trigger mechanism, the frequency of annual failure is related to the frequency of heavy rainfall in the region.

To quantify the vulnerability as a result of landfill slope failure, the ratio of “Factor of Fatality” (FOF) is defined in this study as follow:

$$FoF = \frac{\text{Travel distance}}{\text{Fatal distance}}$$

[4-6]

where “Travel distance” is the horizontal length of waste-flow in failure process, and “Fatal distance” is the horizontal distance of the residential area from landfill site. Figure 4-3 illustrates the corresponding parameters.

![Figure 4-3: The vulnerability parameters of landfill failure](image)

The reports of the fatal landfill failures such as Payatas and Bandung dumpsites confirmed that there is no chance to escape the endangered area during the landfill failure. Therefore, it would be logical to assume that longer “Travel distance” than “Fatal distance” may cause people death in residential area. This fact would happen when the factor of fatality is higher than unity.
Dynamic run-out analysis can provide a good estimation of landslides travel distance. Chapter 3 introduced the proper frictional coefficient in Voellmy rheology to calculate the travel distance of the waste-flow during the failure process. Table 4-2 demonstrated the friction coefficient distributions for different classes of waste type and compaction.

<table>
<thead>
<tr>
<th>Class No.</th>
<th>Waste Compaction</th>
<th>Waste type</th>
<th>Global Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class I</td>
<td>No</td>
<td>Low strength</td>
<td>Mean: 0.12, SD: 0.02</td>
</tr>
<tr>
<td>Class II</td>
<td>No</td>
<td>High strength</td>
<td>Mean: 0.14, SD: 0.02</td>
</tr>
<tr>
<td>Class III</td>
<td>Yes</td>
<td>Low Strength</td>
<td>Mean: 0.16, SD: 0.02</td>
</tr>
<tr>
<td>Class IV</td>
<td>Yes</td>
<td>High Strength</td>
<td>Mean: 0.18, SD: 0.02</td>
</tr>
</tbody>
</table>

Using Taylor series method (Duncan, 2000), the normal distributions of frictional coefficient and the initial friction angle were used as inputs of run-out analysis and the output is the probabilistic distribution of travel distance. Therefore, the application of the amount of “Fatal distance” in each case resulted in the probabilistic distribution of FOF. Considering the FOF distribution, the probability of FOF higher than unity would cause the vulnerability issue in risk assessment of landfill slope failure (P(FOF>1)). Figure 4-4 shows the flowchart of methodology process to calculate special and total risk values.
Figure 4-4: The flowchart of methodology of landfill slope failure risk assessment
4.4 Case Study (Bhalswa Dumpsite in India)

Delhi City has three operational dumpsites to dispose waste materials in Bhalswa, Okhla, and Gazipur. Except the manual compost, there is no technical control over the solid waste management in these sites. Bhalswa dumpsite is located in the North West of Delhi which is operating since 1994. From 1994 to 2011, this dumpsite received about 3000 tons per day. Since 2011, 1200 tons of daily wastes have been diverted to another landfill site Narela Bawana. The site is surrounded by clusters (slum) in East and North, whereas drainage is running parallel to site in the south and west. Figure 4-5 shows the Google map of the dumpsite and the nearby residential area which contained around 25,000 people along the periphery of the site.

Figure 4-5: Bhalswa dumpsite (Google Map, 2014)

The Bhalswa dumpsite has already reached about 40 m of height, and the Steep side slopes are clearly high which in some cross-sections reached to 60 degrees. The main sources of the incoming waste are from the households and the commercial areas. Table 4-3 showed the percentage of the waste composition at Bhalswa dumpsite.
Since, there is no compaction available at the site, and the range of strength waste materials is high, especially stone and bricks components, this dumpsite may be classified as the second class in terms of friction coefficient range. So, the input of run-out analysis (Figure 4-4) in this case study is the normal distribution of friction coefficient with 0.14 for the mean and 0.02 for the standard deviation.

<table>
<thead>
<tr>
<th>Components of Waste</th>
<th>Average Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Biomaterials</td>
<td>50.35</td>
</tr>
<tr>
<td>Paper</td>
<td>3.78</td>
</tr>
<tr>
<td>Plastic</td>
<td>4.45</td>
</tr>
<tr>
<td>Metal</td>
<td>0.1</td>
</tr>
<tr>
<td>Glass and Crockery</td>
<td>1.45</td>
</tr>
<tr>
<td>Ash, stone and bricks</td>
<td>32.7</td>
</tr>
</tbody>
</table>

Table 4-3: Waste composition in Bhalswa dumpsite

In this case study, the frequency of rainfall as an annual trigger mechanism is assumed 0.01.

4.4.1 Case Study Results

One critical cross section beneath the East side of the Bhalswa Landfill is considered to analyse the risk of landfill failure in the adjacent residential area (Figure 4-6). The Landfill is located about 109 m from the residential area. Assuming the translational failures, six failure scenarios on this cross section were analysed with Slope/W and DAN-W programs, providing the probability of failure and the probability of fatality, respectively. Each failure scenario is numbered on Figure 4-6 by the slip surface of that failure scenario. For example, scenario 3 is the failure of all the waste volume in front of the slip surface 3, which includes waste volumes in scenario 1 and 2.

The number of affected elements in this case study assumed one person per meter of failure over the residential area. In other words, if the waste materials during the failure process
progresses towards the residential area, each meter of this accident would kill one person in that residential area.

![Figure 4-6: Six scenarios is compared in terms of landfill slope failure risk](image)

**Table 4-4: Hazard and Consequence parameters for risk evaluation in Bhalswa Landfill cross section**

<table>
<thead>
<tr>
<th>Scenario No.</th>
<th>FoS</th>
<th>P(FoS&lt;1)</th>
<th>FoF</th>
<th>P(FoF&gt;1)</th>
<th>Affected Elements</th>
<th>Special Risk</th>
<th>Total Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.534</td>
<td>94%</td>
<td>0.832</td>
<td>0.01%</td>
<td>3</td>
<td>0.000001</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.569</td>
<td>93.60%</td>
<td>1.053</td>
<td>63.68%</td>
<td>55</td>
<td>0.005960</td>
<td>0.325</td>
</tr>
<tr>
<td>3</td>
<td>0.764</td>
<td>78.38%</td>
<td>1.108</td>
<td>70.19%</td>
<td>79</td>
<td>0.005501</td>
<td>0.434</td>
</tr>
<tr>
<td>4</td>
<td>1.047</td>
<td>42.80%</td>
<td>1.088</td>
<td>62.93%</td>
<td>96</td>
<td>0.002693</td>
<td>0.258</td>
</tr>
<tr>
<td>5</td>
<td>1.405</td>
<td>17.90%</td>
<td>1.044</td>
<td>56.16%</td>
<td>98</td>
<td>0.001005</td>
<td>0.099</td>
</tr>
<tr>
<td>6</td>
<td>1.73</td>
<td>9%</td>
<td>0.997</td>
<td>49.60%</td>
<td>89</td>
<td>0.000437</td>
<td>0.039</td>
</tr>
</tbody>
</table>

As shown in Table 4-4, the lowest factor of safety belongs to the first failure scenario (FOS=0.534), and the probability of this failure obtained 94%. However, the ratio of Factor of Fatality gained 0.832, which means that the failed materials did not reach to the residential area. Thus, the probability of life loss with this scenario is very low (0.01%).

As the distance of slip surfaces from the landfill face are increased, the probabilities of the failures are decreased (94% to 9%). Interestingly, the factor of fatality and the probability of life loss were increased in the first three failure scenarios, and it was decreased when getting further away from the landfill slope surface. According to the calculations, the second failure scenario had the highest special risk, and the highest total risk belongs to the third scenario.
4.4.2 Case Study Discussion

The highest “Hazard” happened in the first failure scenario, while the vulnerability obtained for this situation was zero. In other words, it is possible to experience a high value of hazard and a low value of risk due to the low vulnerability (Quan, 2012). Therefore, it can be inferred that the most probable failure (Lowest factor of safety) is not necessarily the most critical one. The most critical failure may happen in a combination of a high probable failure and a high vulnerability. In this case, the most critical failure happens in the third scenario, because it had the highest total risk in this circumstance.

As the volume of the waste materials, which is failed due to the first failure scenario, was less than the others, so the travel distance of the waste movement during the failure process was less than the fatal distance (109 m) and the probability of life loss was obtained as 0.01%. As the volume of failed waste materials raised for scenario 2 and 3, the vulnerability picked at 70.19%. Higher range of friction between waste materials, within larger volume of failures of scenario 4, 5 and 6 may have led to the reduction in the travel distance of the waste materials in the failure process. Generally, there is an optimum volume of waste materials as a result of which failure, highest travel distance will occur. In this case study, the largest travel distance and the highest vulnerability occurred in scenario 3 in which the highest risk value was obtained.

Since, landfill failure is involuntary risk; Fell (1994) stated that the appropriate amount of the special risk would be less than $10^{-5}$, which is not satisfactory in all the failure scenarios of the case study, except the first one.

4.5 Conclusions

In order to analyse the risk of the landfill slope failure, based on the risk definition, two factors were quantified: Hazard and Vulnerability. To assess the hazard (Probability of landfill failure) of the slope failure for specific cross section of the landfill, the distribution of unit weight, friction angle, and cohesion were used as the input data for the slope stability analysis. Using Taylor series method (Duncan, 2000), the normal distributions of frictional coefficient and
the friction angle were utilized as the inputs of run-out analysis, and the output is the Vulnerability (probability of life loss during the process of landfill failure).

One critical cross section beneath the East side of the Bhalswa Landfill is considered to analyse the risk of the landfill failure in the adjacent residential area. In this case study, six failure scenarios were analysed. The highest “Hazard” happened in the first scenario, while the obtained vulnerability on this scenario was zero. In other words, it is possible to experience a high value of hazard and a low value of risk due to the low vulnerability. Therefore, the most probable failure (Lowest factor of safety) is not necessarily the most critical one. The most critical failure scenario may happen in a combination of a hazardous and vulnerable situation.
Chapter 5
Conclusions and Recommendations

5.1 Conclusions

The development of waste recycling and recovering techniques as well as new treatment methods cannot overcome the landfill requirement to accommodate the residual waste. Due to the environmental challenges in siting new landfills, large landfills with upper heights and steeper slopes are increasingly being adopted in response. Larger landfills pose greater engineering challenges for the slope safety. Stability of MSW landfills depends mainly on the geotechnical characteristics of the waste materials including unit weight, friction angle, cohesion, and pore water pressure. Although it is common to use uniformly constant values for MSW properties in stability analysis, one should consider the spatial and temporal variability and the heterogeneous nature of MSW. In comparison to deterministic stability analysis with constant input values, a probabilistic approach can provide stability analysis considering MSW geotechnical uncertainties.

In this study, the probabilistic distributions for unit weight, friction angle, and cohesion are provided as a result of back-calculation of three dumpsite failures and three landfill failures. In terms of unit weight, the global distribution obtained 11.1 kN/m$^3$ mean value and 3.6 kN/m$^3$ standard deviations, excluding the Bogota landfill failure. The cohesion lognormal distribution of waste materials did not experience major changes with 10 kPa geometric mean value and 3 kPa standard deviation. The mean value for the waste friction angle distribution in the mentioned case studies was 25.4° and the standard deviation of this normal distribution was 8°.
Sudden slope failures of landfills continue to claim lives and destroy properties as well as polluting the environment (e.g., surface and ground water). Waste travel distance following a landfill failure is crucially important to estimate in order to calculate the extent of endangered area and the potential vulnerability of the elements at risk. The travel distance of the waste can be calculated through back-analysing a set of case studies using run-out modeling, then applying the optimized parameters in forward modeling. Statistical back-analyses of Voellmy rheology performance in the case studies of dumpsite failures and landfill failures are provided. Waste characteristics, including compaction and material types, are concluded to affect frictional parameters of Voellmy rheology. Waste compaction on landfills may increase the friction coefficient rather than dumpsites. In addition, in the absence of high strength waste materials such as wood, metal, plastic, paper, and cardboard, the friction is destructively reduced. In this case, for initial forward modeling, four classes of friction global distributions were recommended for run-out analysis of waste-flow given the potential for landfill failure. The distributions of friction coefficient with 0.12 and 0.14 mean values are appropriate for uncompacted waste with low and high strength components, respectively. Furthermore, 0.16 and 0.18 are suitable mean values for compacted waste with low and high strength components, respectively. These distributions can be used to simulate waste-flow during the landfill failure process.

Geotechnical design based on factor of safety is the most popular method to analyse the slope stability of landfills. However, catastrophic landfill and dumpsite failures have shown that considering the lowest factor of safety (only hazard) is not sufficient for designing, and the probable vulnerability as a result of failure may have to be considered, as well. This study combined hazard and vulnerability to design landfill slope stability, providing the waste slope stability parameters in probabilistic approach to analyse the probability of failure (hazard), and the probabilistic waste rheology data to analyse the probability of fatality (vulnerability), the framework of risk evaluation for landfill slope stability obtained. This method was used to evaluate the risk in an Indian landfill as a case study.

One critical cross section beneath the East side of the Bhalswa Landfill is considered to analyse the risk of landfill failure in the adjacent residential area. In this case study, six failure scenarios were analysed. The highest “hazard” happened in the first scenario, while the
“vulnerability” on this scenario obtained zero. In other words, it is possible to experience a high value of hazard and a low value of risk due to the low vulnerability. So, the most probable failure (Lowest factor of safety) is not necessarily the most critical one. The most critical failure scenario may happen in a combination of hazardous and vulnerable situation.

5.2 Application of Knowledge Learned

Regarding the landfill slope stability design, risk assessment using the Probabilistic approach can be potentially the alternative method to factor of safety evaluation (deterministic approach). In order to analyse the risk of the landfill slope failure, two factors involved are: Hazard and Vulnerability.

In terms of the “Hazard”, the required data for analysis of the slope stability is landfill geometry (height and slope) and waste geotechnical characteristics. This study (chapter 2) provided the probabilistic distributions for waste unit weight, cohesion, and friction angle (Table 5-1). These results can be used for project-specific design when data are not available.

<table>
<thead>
<tr>
<th>Waste characteristics</th>
<th>Distribution Type</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight</td>
<td>Normal</td>
<td>11.1 kN/m³</td>
<td>3.6 kN/m³</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>Normal</td>
<td>25.4°</td>
<td>8°</td>
</tr>
<tr>
<td>Cohesion</td>
<td>Lognormal</td>
<td>10 kPa</td>
<td>3 kPa</td>
</tr>
</tbody>
</table>

To assess the hazard of the slope failure for specific cross section of the landfill, the developed probability distributions from this study can be used as the input data for the slope stability analysis. Using Monte-Carlo method or Taylor Series method, the probability distribution of factor of safety can be computed. The slope failure can happen only if the factor of safety was lower than unity which is the probability of failure ($P(FOS<1)$). The probability can be calculated for safe slopes (1:3 or 1:4) with different heights of the landfill to use in slope stability design.

In terms of “Vulnerability”, the required data for analysis of the landfill failure fatality includes the distance of landfill from vulnerable area (fatal distance) and waste rheological
characteristics. This study (chapter 3) provided the probabilistic distribution for waste frictional coefficient in Voellmy rheology based on waste compaction and waste type (Table 5-2).

Table 5-2: Friction coefficient distributions based on waste compaction and waste type

<table>
<thead>
<tr>
<th>Class No.</th>
<th>Waste Compaction</th>
<th>Waste type</th>
<th>Global Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>Class I</td>
<td>No</td>
<td>Low strength</td>
<td>0.12</td>
</tr>
<tr>
<td>Class II</td>
<td>No</td>
<td>High strength</td>
<td>0.14</td>
</tr>
<tr>
<td>Class III</td>
<td>Yes</td>
<td>Low Strength</td>
<td>0.16</td>
</tr>
<tr>
<td>Class IV</td>
<td>Yes</td>
<td>High Strength</td>
<td>0.18</td>
</tr>
</tbody>
</table>

To assess the vulnerability of the slope failure for specific cross section of the landfill, appropriate class of the friction coefficient distribution (Table 5-2) as well as friction angle distribution (Table 5-1), obtained through this Masters Research thesis, can be used as the input data for the run-out analysis. Using Monte-Carlo method or Taylor Series method, the probability distribution of travel distance can be computed. With the application of “Fatal distance” the probabilistic distribution of FOF can be obtained. Considering the FOF distribution, the probability of FOF higher than unity would cause the vulnerability issue in risk assessment of landfill slope failure (P(FOF>1)). This probability can be calculated for safe slopes (1:3 or 1:4) with different heights of the landfill and will help in siting the landfill at a safer distance away from residential area or will help in designing a secondary containment unit (e.g., a barricade wall) between the landfill site and the vulnerable area (e.g., residential area, structure of importance etc.).

Considering heavy rainfall as a trigger mechanism of the landfill failure, special risk would be equal to:

\[
\text{Risk} = P(FOS<1) \times P(FOF>1) \times \text{Rainfall frequency}
\]  

[5-1]

Since, landfill failure is involuntary risk; Fell (1994) stated that the safe value of the special risk is less than \(10^{-5}\). Comparing the risk values (based on hazard and vulnerability probabilities) for different landfill heights, the safe height of the landfill would be obtainable with equation [5-1].

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5.3 Future Research and Recommendations

The future research on landfill failure slope stability, failure mobility and risk analysis can be focused on different aspects summarized as follows:

1. In future studies, the shear strength parameters (friction angle and cohesion) may be studied as dependent factors to each other, which may provide the combined probabilistic distribution rather than independent distributions in this study.

2. In future studies, the mobility and slope stability analyses as well as risk calculations can be provided with three dimensional programs which probably more accurate for landfill design.

3. In this study, the risk calculation and assessment concentrated on human life loss. However, the environmental surface and the ground water pollutions as well as properties are potential vulnerable parameters that require more research to analyse the risk.

4. In this study, the same unit weight, friction angle and cohesion distributions of the waste materials have been used for different spatial and temporal situations. In future studies, it would be valuable to classify the range of these parameters for different landfill situations, such as old or fresh waste, compacted or uncompacted waste, high and low normal stress in landfill, etc.

5. Although this research focused on Landfill failure risk analysis, this tool would be applicable for other landslide categories such as avalanche, debris-flow and rock-flow.
Appendices
Appendix A

Zhang et al. (2011) mathematic equation to back analyse normally distributed parameters in slope stability analyses:

Assuming $\mu_\theta$ as a mean and $C_\theta$ as a covariance matrix of $\theta$ primary distribution, the objective of probabilistic back-analysis is then to improve probabilistic distribution of $\theta$ to $\mu_{(\theta|d)}$ and $C_{(\theta|d)}$:

$$
\mu_{(\theta|d)} = \mu_\theta + C_\theta H^T (H C_\theta H^T + \sigma_\varepsilon^2)^{-1} [1 - g(\mu_\theta) - \mu_\varepsilon] \tag{A-1}
$$

$$
C_{(\theta|d)} = \left( \frac{H^T H}{\sigma_\varepsilon^2} + C_\theta^{-1} \right)^{-1} \tag{A-2}
$$

$$
H = \left. \frac{\partial g(\theta)}{\partial \theta} \right|_{\theta=\mu_\theta} \tag{A-3}
$$

Where $g(\mu_\theta) = $ predicted factor of safety calculated at point $\mu_\theta$ and $H = $ row vector representing the sensitivity of $g(\theta)$ with respect to $\theta$ at point $\mu_\theta$. $E$ is defined to quantify the effect of model imperfection. So, $\mu_\varepsilon$ and $\sigma_\varepsilon$ are the mean and standard deviation of this factor. Since, Bishop method is utilized in this study, $\mu_\varepsilon = 0.05$ and $\sigma_\varepsilon = 0.07$ are suggested by Christian et al. (1994).

The limitation of this method is that it is applicable only when $g(\theta)$ is largely linear around point $\mu_\theta$. 
## Appendix B

### Table B-1: The literature data for Unit weight of MSW

<table>
<thead>
<tr>
<th>Author</th>
<th>Year</th>
<th>Unit weight (kN/m$^3$)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landva &amp; Clark</td>
<td>1990</td>
<td>10 to 14</td>
<td>Dry density*</td>
</tr>
<tr>
<td>Richardson &amp; Reynolds</td>
<td>1991</td>
<td>16</td>
<td>Dry density</td>
</tr>
<tr>
<td>Withiam et al.</td>
<td>1995</td>
<td>10.8 to 12.8</td>
<td>Dry density</td>
</tr>
<tr>
<td>Edincilier et al.</td>
<td>1996</td>
<td>7.5 to 14.2</td>
<td>Dry density</td>
</tr>
<tr>
<td>Siegel et al.</td>
<td>1990</td>
<td>9.6 to 17.3</td>
<td>Dry density</td>
</tr>
<tr>
<td>Del Greco &amp; Oggeri</td>
<td>1994</td>
<td>5 to 7</td>
<td>Dry density</td>
</tr>
<tr>
<td>Gabr &amp; Valero</td>
<td>1995</td>
<td>10 to 12.1</td>
<td>Dry density</td>
</tr>
<tr>
<td>Mazzucato et al.</td>
<td>1999</td>
<td>7</td>
<td>Bulk density*</td>
</tr>
<tr>
<td>Thomas et al.</td>
<td>1999</td>
<td>7.8 to 16</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Pelkey et al.</td>
<td>2001</td>
<td>10 to 16</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Machado et al.</td>
<td>2002</td>
<td>10</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Vilar &amp; Carvalho</td>
<td>2004</td>
<td>10 to 12</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Gomes et al.</td>
<td>2005</td>
<td>11.5</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Itoh et al.</td>
<td>2005</td>
<td>6 to 7</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Harris et al.</td>
<td>2006</td>
<td>11 to 17.5</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Isenberg</td>
<td>2003</td>
<td>16</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Caicedo et al.</td>
<td>2002</td>
<td>10</td>
<td>Bulk density</td>
</tr>
<tr>
<td>Watts and Charles</td>
<td>1990</td>
<td>6 to 8</td>
<td>moderate compaction</td>
</tr>
<tr>
<td>Manassero et al.</td>
<td>1996</td>
<td>5 to 10</td>
<td>moderate compaction</td>
</tr>
<tr>
<td>Gourc et al.</td>
<td>2001</td>
<td>7</td>
<td>upper layers, Fresh</td>
</tr>
<tr>
<td>Kavazanjian et al.</td>
<td>2001</td>
<td>6 to 7</td>
<td>upper layers, Fresh</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>down layers, old with soil type components</td>
</tr>
<tr>
<td>Matasovic et al.</td>
<td>2008</td>
<td>15</td>
<td>6 meter depth</td>
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<tr>
<td>Singh et al.</td>
<td>2009</td>
<td>8.8 to 10.1</td>
<td>Intact</td>
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<td></td>
<td></td>
<td>11.3 to 13.7</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>12.6 to 15.8</td>
<td>Re-compacted</td>
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<tr>
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<td>1995</td>
<td>3.3</td>
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<td></td>
<td></td>
<td>12.8</td>
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<tr>
<td>Fassett et al.</td>
<td>1994</td>
<td>8 to 14</td>
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<td>Yanru et al.</td>
<td>2012</td>
<td>10.22</td>
<td>unshredded landfill samples</td>
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<td>Zhan et al.</td>
<td>2008</td>
<td>11</td>
<td>Cone penetration test</td>
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<tr>
<td>Huvaj-sarihan et al.</td>
<td>2008</td>
<td>11</td>
<td>Assumed for back analyses</td>
</tr>
<tr>
<td>Stark &amp; Eid</td>
<td>2000</td>
<td>10.2</td>
<td>Used for back analyse</td>
</tr>
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</table>

*Dry, bulk and saturated density are three different forms of unit weight evaluation. Comment column contains the major effective factors on waste unit weight for each specific case. Excluding last two row of the table, the rest of unit weights are the results of experimental studies.*
<table>
<thead>
<tr>
<th>Author</th>
<th>Year</th>
<th>Friction Angle (°)</th>
<th>Cohesion (kPa)</th>
<th>Dimensions (mm)</th>
<th>Strain</th>
<th>Comment</th>
</tr>
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<tbody>
<tr>
<td>Siegel et al.</td>
<td>1990</td>
<td>39</td>
<td>0</td>
<td>d=130</td>
<td>10%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>53</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landva et al.</td>
<td>1990</td>
<td>24</td>
<td>23</td>
<td>N/A</td>
<td>N/A</td>
<td>Fresh, Shredded MSW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>39</td>
<td>19</td>
<td>N/A</td>
<td></td>
<td>Fresh, Typical MSW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>16</td>
<td></td>
<td></td>
<td>8 years old, Artificial MSW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>0</td>
<td></td>
<td></td>
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<td></td>
<td>41</td>
<td>0</td>
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<tr>
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<td></td>
<td>36</td>
<td>0</td>
<td></td>
<td></td>
<td>Old, Typical MSW</td>
</tr>
<tr>
<td>Landva et al.</td>
<td>1990</td>
<td>42</td>
<td>19</td>
<td>434*287</td>
<td>Peak</td>
<td>Fresh</td>
</tr>
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<td></td>
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</tr>
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<td>33.6</td>
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<tr>
<td>Del Greco et al.</td>
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<td>21</td>
<td>15.7</td>
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<td>N/A</td>
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<td>22</td>
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<td>0</td>
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<tr>
<td>Houston et al.</td>
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<td>34</td>
<td>5</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>Withiam</td>
<td>1995</td>
<td>30</td>
<td>10</td>
<td>1500*1500</td>
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<td>Gabr et al.</td>
<td>1995</td>
<td>20 to 39</td>
<td>0 to 28</td>
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<td>10%-20%</td>
<td>15-30 years old, Typical MSW</td>
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<td>Kavazanjian</td>
<td>1995</td>
<td>31</td>
<td>43</td>
<td>d=460</td>
<td>15%</td>
<td>11 to 35 years old, Wood Waste</td>
</tr>
<tr>
<td>Edincliler</td>
<td>1996</td>
<td>42</td>
<td>27</td>
<td>d=300</td>
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<td>43</td>
<td>d=800</td>
<td>Peak</td>
<td>Artificial MSW</td>
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<tr>
<td>Eid et al.</td>
<td>2000</td>
<td>42</td>
<td>25</td>
<td>N/A</td>
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<td>2001</td>
<td>17.8</td>
<td>21</td>
<td>300*450</td>
<td>25 mm</td>
<td>Fresh</td>
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<td>21.8</td>
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<td>28</td>
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<td></td>
</tr>
<tr>
<td>Caicedo</td>
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<td>23</td>
<td>67</td>
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<td>1 year old</td>
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<td>2002</td>
<td>24-32</td>
<td>N/A</td>
<td>d=101</td>
<td>N/A</td>
<td>Fresh</td>
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<tr>
<td>Mahler et al.</td>
<td>2003</td>
<td>36</td>
<td>4</td>
<td>400*250</td>
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<td></td>
<td></td>
<td>21</td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gabr et al.</td>
<td>2007</td>
<td>32</td>
<td>N/A</td>
<td>d=100</td>
<td>10%</td>
<td>24 days</td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
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<td></td>
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<td>24</td>
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<td>127 days</td>
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<tr>
<td>Reddy et al.</td>
<td>2008</td>
<td>29</td>
<td>47</td>
<td>900*1500</td>
<td>15%</td>
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<td>Author</td>
<td>Year</td>
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<td>Ruddy et al.</td>
<td>2009a</td>
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<tr>
<td>Reddy et al.</td>
<td>2009b</td>
<td>30</td>
<td>46</td>
<td>d=63.5</td>
<td>15%</td>
<td>Fresh</td>
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</tr>
<tr>
<td>Singh et al.</td>
<td>2009</td>
<td>36</td>
<td>14</td>
<td>300*450</td>
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</tr>
<tr>
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<tr>
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<td>35</td>
<td>1</td>
<td>d=63.5</td>
<td>15%</td>
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<td>16</td>
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<td>40</td>
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</tr>
<tr>
<td>Yanru et al.</td>
<td>2011</td>
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<td>d=300</td>
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<td>older than 10 years</td>
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<td>8.4</td>
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<tr>
<td>Yanru et al.</td>
<td>2011</td>
<td>24.4</td>
<td>4.9</td>
<td>d=300</td>
<td>15%</td>
<td>Fresh</td>
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<td>4.5</td>
<td></td>
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## Table B-3: Triaxial test data for MSW

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<th>Year</th>
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<th>Cohesion (kPa)</th>
<th>Dimensions (mm)</th>
<th>Strain</th>
<th>Comment</th>
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<td>Stoll</td>
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<td>24 to 42</td>
<td>0</td>
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<td>N/A</td>
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<tr>
<td>Gabr and Valero</td>
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<td>34</td>
<td>17</td>
<td>d=71</td>
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<td></td>
</tr>
<tr>
<td>Jessberger et al.</td>
<td>1995</td>
<td>31 to 49</td>
<td>N/A</td>
<td>d=300</td>
<td>N/A</td>
<td></td>
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<tr>
<td>Grisollia et al.</td>
<td>1995</td>
<td>15 to 20</td>
<td>2 to 3</td>
<td>d=300</td>
<td>10%-15%</td>
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<tr>
<td>Aburatani</td>
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<td>30 to 35</td>
<td>N/A</td>
<td>d=300</td>
<td>N/A</td>
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<tr>
<td>Caicedo</td>
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<td>45</td>
<td>14</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>Vilar et al.</td>
<td>2002</td>
<td>23.3</td>
<td>20</td>
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<tr>
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<td>32</td>
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<tr>
<td>Machado et al.</td>
<td>2002</td>
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<td>d=200</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>16.5 to 20</td>
<td>25 to 30</td>
<td></td>
<td>10%</td>
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<tr>
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<td>21 to 27</td>
<td>65 to 70</td>
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<tr>
<td></td>
<td></td>
<td>16.5 to 18</td>
<td>0.5 to 6</td>
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<tr>
<td></td>
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<tr>
<td>Feng et al.</td>
<td>2005</td>
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<td>4 to 7</td>
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<td>5 years old</td>
</tr>
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<td>14 to 17</td>
<td>15 to 28</td>
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</tr>
<tr>
<td></td>
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<td>17 to 19</td>
<td>30 to 55</td>
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<td>15%</td>
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</tr>
<tr>
<td>Vilar et al.</td>
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<td>22</td>
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<td>d=400</td>
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<td>33</td>
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<tr>
<td>Zekkos et al.</td>
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<td>36 to 41</td>
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<td>N/A</td>
<td>N/A</td>
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<tr>
<td>Zwanenbureg et al.</td>
<td>2007</td>
<td>35 to 37</td>
<td>0</td>
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<td>N/A</td>
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<tr>
<td>Zhan et al.</td>
<td>2008</td>
<td>9.9</td>
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<td></td>
<td>39</td>
<td>0</td>
<td></td>
<td>11 years old</td>
<td></td>
</tr>
<tr>
<td>Reddy et al.</td>
<td>2008</td>
<td>12</td>
<td>32</td>
<td>50</td>
<td>15%</td>
<td>Fresh to complete degraded</td>
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<td></td>
<td></td>
<td>16</td>
<td>38</td>
<td></td>
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</tr>
<tr>
<td>Singh et al.</td>
<td>2009</td>
<td>35 to 47</td>
<td>0 to 8.4</td>
<td>d=150</td>
<td>N/A</td>
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<tr>
<td>Reddy et al.</td>
<td>2011</td>
<td>1 to 11</td>
<td>18 to 56</td>
<td>50</td>
<td>15%</td>
<td>Fresh to complete degraded</td>
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<tr>
<td>Yanru et al.</td>
<td>2011</td>
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<td>35.9</td>
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<td>15%</td>
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<td></td>
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<td>37</td>
<td>66.4</td>
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Table B-4: Simple Shear test for MSW

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<th>Cohesion (kPa)</th>
<th>Dimensions (mm)</th>
<th>Strain</th>
<th>Comment</th>
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<td>42</td>
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<td>N/A</td>
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<td>Gay and Kaiser</td>
<td>1981</td>
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<td>28</td>
<td>N/A</td>
<td>N/A</td>
<td>fresh</td>
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<tr>
<td>Jessberger</td>
<td>1994</td>
<td>42</td>
<td>7</td>
<td>N/A</td>
<td>N/A</td>
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<td>Taylor</td>
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<td>31</td>
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<td>N/A</td>
<td>10%</td>
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<tr>
<td>Kavazanjian</td>
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<td>33</td>
<td>16</td>
<td>d=454</td>
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<td>Pelkey et al.</td>
<td>2001</td>
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Table B-5: Back analysed friction angle and cohesion data for MSW

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<th>Year</th>
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<th>Cohesion (kPa)</th>
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<td>Spillman</td>
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<td>10</td>
</tr>
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<td>1980</td>
<td>17</td>
<td>10</td>
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<tr>
<td>Cowland et al.</td>
<td>1993</td>
<td>25</td>
<td>10</td>
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<tr>
<td>Kavazanjian</td>
<td>1995</td>
<td>33</td>
<td>24</td>
</tr>
<tr>
<td>Stark</td>
<td>2000</td>
<td>35</td>
<td>40</td>
</tr>
<tr>
<td>Merry</td>
<td>2005</td>
<td>28</td>
<td>19</td>
</tr>
<tr>
<td>Koelsch</td>
<td>2005</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Blight</td>
<td>2008</td>
<td>35</td>
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Table B-6: Suggested friction angle and cohesion data for MSW

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<th>Situation</th>
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<td>1994</td>
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<td>0</td>
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<td>Fassett et al.</td>
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<td>10</td>
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<td>Koelsch</td>
<td>1995</td>
<td>22</td>
<td>18</td>
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<td></td>
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<td>15</td>
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<tr>
<td>Kavazanjian et al.</td>
<td>1995</td>
<td>0</td>
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<td>33</td>
<td>0</td>
<td>Higher than 30 kPa normal stress till 300</td>
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<tr>
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<td>35 to 40</td>
<td>1 to 2</td>
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<td>1998</td>
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<td>less than 20 kPa normal stress</td>
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<td>Van Impe</td>
<td>1998</td>
<td>30</td>
<td>20</td>
<td>higher than 60 kPa normal stress</td>
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<tr>
<td>Eid et al.</td>
<td>2000</td>
<td>35</td>
<td>25</td>
<td>limited to normal stresses less than 350 kPa</td>
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<tr>
<td>Isenberg</td>
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<td>20 to 35</td>
<td>0 to 50</td>
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<td>stark</td>
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<td>35</td>
<td>6</td>
<td>less than 200 kPa normal stress</td>
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<td>30</td>
<td>Higher than 200 kPa normal stress</td>
</tr>
<tr>
<td>Bray</td>
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<td>36</td>
<td>15</td>
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Appendix C

Assuming friction coefficient and initial friction angle distributions as input ($IP_1$, $IP_2$) and travel distance distribution as an output (OP) of the model, Taylor’s series method can be used to calculate the mean value and standard deviation of the output distribution. The mean value of travel distance distribution can be obtained considering the mean value for friction angle and friction coefficient parameters. The standard deviation of travel distance distribution is equal to:

$$SD = \sqrt{\left(\frac{\Delta OP_1}{2}\right)^2 + \left(\frac{\Delta OP_2}{2}\right)^2}, \quad [C-1]$$

where $\Delta OP_1$ and $\Delta OP_2$ can be provided by increased and decreased $IP_1$ and $IP_2$ mean values by one standard deviation (Duncan, 2000).

As a result, the friction angle and friction coefficient distributions conducted to travel distance distribution. This may beneficial to find the range of waste movement when the landfill failed.
References


Proceedings Sardinia 99, seventh international waste management and landfill symposium, Cagliari, Italy


