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2D and 3D Back Analysis of the Forest City Landslide (South Dakota)

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2D and 3D Back analysis of Forest City (South Dakota) landslide

by

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ABSTRACT

Landslides are a common geologic feature in the Missouri River trench and along the valleys of Missouri River tributaries. These landslides are commonly found to develop in the Pierre Shale formation in this region. Pierre Shale is a weakly cemented marine clay shale developed in Cretaceous age by sediments from Epeiric Sea. This clay shale is well known for imposing engineering challenges in the form of slope instability and foundation difficulties because of its non-homogeneity and high plasticity. It is known as heavily overconsolidated shale which can fail due to minor disturbances. Based on the development of dams and transportation infrastructure in this area understanding the behavior of Pierre shale is extremely important to assess the stability of those structures. As the shale material in this area is already fissured and has the history of numerous landslides, its residual strength is considered over peak strength to efficiently represent its strength. This thesis investigates a possible range of the residual friction angle for the Pierre Shale. The Forest City landslide which occurred in the Missouri River trench is selected as a case study for this purpose. The residual friction angle values are evaluated by performing a deterministic back analysis of the slope in two and three dimensions. The deterministic two dimension analysis is performed in limit equilibrium and finite element methods using SLOPE/W and SIGMA/W softwares from GEOSTUDIO 2007. A deterministic three dimensional analysis is performed by using CLARA/W software. The values obtained from these analyses are compared and a reasonable value of 4° to 6.64° is selected to represent the residual friction angle values for the Pierre Shale.

CHAPTER 1. INTRODUCTION

Landslides are a common geologic feature in the Missouri River trench, South Dakota. This river trench splits the state of South Dakota into two - east and west parts. The formation of this river trench about 135,000 to 70,000 years ago by melting of glaciers caused tremendous unloading in the nearby surfaces. This unloading effect resulted in the formation of a heavily overconsolidated clay shale material, termed as Pierre shale. Landslide activity in the Missouri river trench and along its tributaries is primarily found in the Pierre shale formation. The development of dams and transportation infrastructure such as highways and bridges in this region has resulted in initiating numerous studies about the influence of behavior of the Pierre Shale on the stability of these structures. An important aspect in understanding the behavior Pierre Shale is to know its geologic history which resulted in the formation of landslides and fractures in this area. Based on the geologic history of landslides in this region, understanding the residual shear strength behavior of Pierre shale is a key aspect. The peak shear strengths are generally estimated only in cases of first time failure of a slope. Since the geologic history of Pierre Shale shows that movements have already occurred in the Pierre shale formation, residual strengths are considered to represent its strength. Numerous examples are available in the literature which specifies that after clay soil has been subjected to large amount of shear strains its strength is reduced to residual value (Morgenstern 1977 and Skempton 1985). Knowledge about the residual strength of Pierre shale is particularly important because any small disturbances to this clay shale can cause the historic landslides to reactivate (Brooker and Peck 1993).

Residual shear strengths can be determined by performing a back analysis of a landslide or by conducting laboratory tests. The effectiveness of the residual strength obtained from performing a back analysis is highly dependent on the efficiency of failure surface defined, geometry of the slope, soil model selected and the type of analysis performed. In this thesis, an attempt to find the possible range of the residual shear strength of Pierre shale is made by conducting a back analysis for the case of Forest City landslide along the Missouri River.

1.1 Problem Statement

The main aim of this thesis is to investigate a possible range for the residual shear strength of Pierre Shale layer involved in the Forest City Landslide, South Dakota. This landslide is located in the Missouri River trench. To estimate the residual strength a back analysis of the Forest City landslide has been conducted. The back analysis has been conducted considering two dimensional and three dimensional effects of the slope in a limit equilibrium frame work to understand the variation of the strength obtained between 2D and 3D analysis. Also, a 2D finite element analysis was conducted considering the stress-strain behavior of the soil layers involved. Based on these analyses results a possible range of the residual strength has been provided.

1.2 Organization of the Thesis

The entire thesis is divided into five chapters along with an appendix included at the end of the thesis. The first chapter consists of introduction. The second chapter is a review of literature of the Pierre Shale background, limit equilibrium and finite element slope stability analysis methods and extension of its mechanics to three dimensions etc. The third chapter explicitly describes about the Forest City landslide and its geologic history. Chapter four consists of the 2D and 3D deterministic analysis results and discussion of the results obtained. The last chapter provides conclusions and the scope for further research. This chapter is followed by appendix.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

This section of the thesis provides a brief review of the methods used for two dimensional (2D) and three dimensional (3D) slope stability analysis. A comparison among the available 2D Limit Equilibrium methods (LEM) is reviewed along with a brief discussion on the extension of 2D method of slices to 3D method of columns. Background information about the soil layer - the Pierre Shale (weathered) in which the failure occurred is provided in this section and the details about the landslide are mentioned in the next section. Also, the simulation softwares used for 2D and 3D analyses are discussed here with an explanation provided for the type of soil models used.

2.2 Slope Stability Analyses

Slope stability analyses are mainly performed to assess the safety factor of a particular slope in a given geologic and physical conditions. For a slope to be stable the resisting forces in the slope must be sufficiently greater than the forces causing the failure (Duncan and Wright 2005). Stability analysis can be used for the following,

1) To assess the safety of a structure in terms of its stability.
2) To locate the critical failure surface and to know it shape of failure.
3) To understand and numerically evaluate the sensitivity of stability to its geologic parameters and climatic conditions.
4) To assess the movement of the slope.
5) To assess remedial measures and aid in their design.

To perform a slope stability analysis the geometry of the slope, external and internal loading, soil stratigraphy and strength parameters and variation of the ground water table all along the slope must be defined. In the current state of practice, there are many number of slope stability analysis methods available. However, the scope of this report is limited to a discussion on the limit equilibrium methods and finite element methods.

2.2.1 Limit equilibrium method

The limit equilibrium method of analysis is a well-established method and widely used by the geotechnical engineers. This method mainly provides an assessment of stability of the slope in terms of its safety factor. For determining the factor of safety of a particular slope the primary requirement is the strength properties of the soil material involved and does not consider its stress – strain behavior. The limit equilibrium method provides only an estimate of the stability of a slope but doesn't provide any information about the magnitude of movement of the slope.

2.2.1.1 Mechanics of two dimensional (2D) limit equilibrium analyses

In limit equilibrium techniques, the slope stability is assessed by calculating the factor of safety of a slope. This value is determined for an infinite number of slip surfaces, but the value obtained for the most critical failure surface (failure surface with the least factor of safety value), termed as Critical / Minimum Factor of safety signifies the stability. A Factor of safety value is defined as the ratio of available shear strength (s) in a slope to its equilibrium shear stress (τ) i.e., the strength factor required just to maintain the stability of the slope (Duncan and Wright 2005). The factor of safety definition is represented in Equation 2-1.

$$F = s / \tau \tag{2-1}$$

In this method, most of the slope stability analyses methods are statically indeterminate and assumptions about the distribution of internal forces acting on them are required to solve this redundancy. The assumptions differ for each limit equilibrium method. The factor of safety is found by the application of force and/or moment equilibrium. The static limit equilibrium methods have two different approaches (1) Single Free Body Procedures and (2) Method of Slices. In the Single Free body procedures the entire mass of the soil is considered to be in equilibrium and a single free body diagram is assumed for the entire mass. The infinite slope method, Swedish slip circle method and logarithmic spiral method are some of the examples of these methods. But this method imposes challenges in calculations when used in case of a non-circular or a wedged slip surface. As most of the landslides observed in the real world fail with a non-circular failure surface, this method was not popularly used. The method of slices was adopted by many geotechnical engineers to overcome this disadvantage. The method of slices is appropriate to solve both circular and non-circular slip surfaces separately (Duncan and Wright 2005). In this method, the entire slip surface is divided into number of vertical slices and equilibrium equations are applied to each slice.. This method is illustrated in Figure 2-1. The figure represents a circular slip surface which is subdivided into slices and also a single slice (named as i^{th} slice in the figure) is shown with forces acting on it. For illustration purpose, the slice forces used in Bishop's Simplified method are shown. The forces on the slices vary from one method to another method. In this figure, W_i represents the weight of the i^{th} slice, Si is the shear force at the base of the i^{th} slice, a_i is the moment arm, α_i is the inclination of the base of the slice. In the single slice figure, N represents the normal force acting in the base of the slice, E_i and E_{i+1} represent the forces acting on the slides of the slices (shear stresses in between the slices are neglected in Simplified Bishop's method). So, using these forces, resisting and driving moments are calculated and their ratio gives the factor of safety value.

Some of the popular methods which follow the procedure of slices are the Ordinary Method of Slices (Swedish method of slices or Fellenius method 1927), Bishop's Simplified procedure (Bishop 1955), Janbu's method (1973), Spencer's Method (1967), Morgenstern and Price Method (1965). Each of these methods do not satisfy all the three static equilibrium conditions of 1) equilibrium of forces in vertical direction, 2) equilibrium of forces in horizontal direction and 3) equilibrium of moments about any point. Hence, different assumptions are made for each procedure to get a balance of known equations and unknown quantities. The side force assumption is one of the main characteristics which distinguishes one limit equilibrium method from another (Griffiths and Lane 1999).

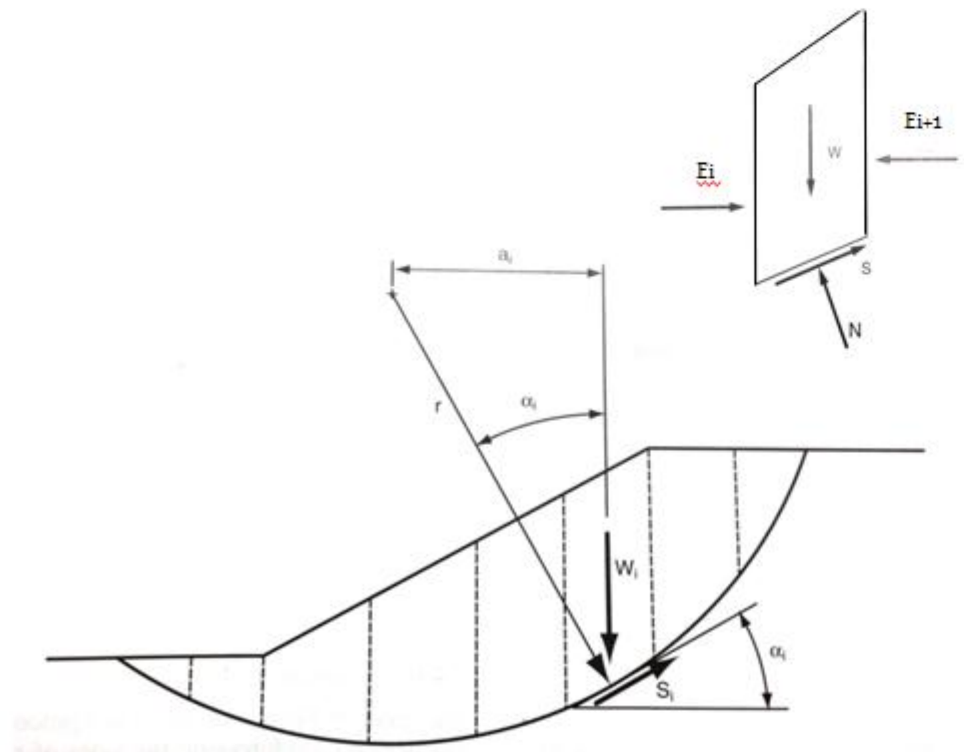

Figure 2-1: Typical representation of a circular slip surface subdivided into vertical slices and forces acting on it (adopted from Duncan and Wright 2005).

Accuracy of the computational methods available is based on the extent to which it can satisfy the equilibrium conditions and its assumption on the inclination of side forces on each slice. According to Duncan and Wright (2005), the accuracy of the methods is described in Table 2-1.

The maximum variation in the factor of safety values obtained by using the above mentioned methods taking into account their limitations is ± 6% (Duncan and Wright 2005). As the Morgenstern – Price and the Spencer's method satisfy both the moment and force equilibrium they are considered to be most accurate methods (Duncan and Wright 2005). Also, it is observed that both the methods result in identical factor of safety values (Duncan and Wright 2005 & Fredlund and Krahn 1977). Based on the accuracy for each method discussed, only Morgenstern-Price, Spencer's method and Bishop's method are used for conducting back analysis of the Forest City landslide in 2D in the further sections.

Table 2-1: Summary of 2D Limit Equilibrium methods for Slope stability analysis (after, Duncan and Wright (2005))

Method	Accuracy and Limitations
Ordinary method of slices (Fellenius 1927)	• Gives a very low Factor of safety value in case of effective stress analyses for flat slopes with high pore water pressures. • Accurate only when Ø = 0 analyses • Accurate in case of total stress analyses with circular slip surfaces.
Modified Swedish method (Corps of Engineers 1970)	• Applicable for all types of slip surfaces • Factor of safety values are generally higher than the other methods which satisfy all the conditions of equilibrium.
Bishop's modified method (Bishop 1955)	• Accurate only when circular slip surfaces are involved. • Factor of safety values differ 3% to 5% from the Ordinary method of slices.
Janbu's simplified method (Janbu 1968)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface • Results in a lower factor safety values than other methods satisfying all equilibrium equations.
Spencer's method (Spencer 1967)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface
Morgenstern and Price method (Morgenstern and Price 1965)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface

2.2.2 Finite element method

The finite element method was first introduced to geotechnical engineers in 1966 Berkeley conference on stability of slopes and embankments by Clough and Woodward (1967). Unlike the limit equilibrium method, the finite element method considers linear and non-linear stress – strain behavior of the soil in calculating the shear stress for the analysis. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffiths and Lane 1999).

Finite element methods are well known for the estimating the realistic deformations of the slopes and embankments. Some of the advantages of using a finite element analysis over limit equilibrium methods are,

1) The movement of the slopes at a particular location can be calculated. This helps in monitoring the movement of the slope. Also, soil stresses and pore water pressure responses to different external factors such as load, water level, reservoir level etc. can be calculated.
2) Stability of the slope during staged construction such as step by step excavation or construction of embankments, levees etc. can be calculated by performing incremental analysis.

The types of soil stress-strain relationships that can be used are linear elastic, elastoplastic, hyperbolic, Modified Cam Clay, elastoviscoplastic and multilinear elastic models. The selection of a particular stress-strain relationship depends on the state of the soil structure to be analyzed, its purpose of analysis and its laboratory and field properties available. The determination of soil properties in the field involves a large amount of uncertainty and so the application of finite element analyses imposes complexity on the stability problem (Griffiths and Lane 1999).

Traditionally, the slope stability analysis with a finite element approach is performed by Strength reduction method (SRM). In this method, the factor safety is defined as the factor by which the original shear strength parameters must be divided to bring the slope to be in failure mode (Griffiths and Lane 1999). Hence, the factor shear strength parameters (c'_f and $Ø'_f$) are shown as follows,

$$c'_f = c' / SRF \qquad \textbf{2-2}$$

$$Ø'_f = \arctan (\tan Ø' / SRF) \qquad \textbf{2-3}$$

Where SRF is the Strength Reduction Factor. A systematic estimation is required for the SRF value to find out the value which will just cause the slope to fail. The SRF value, at which the slope will just to fail, is known as the factor of safety. The failure condition in this method could be when 1) the non-linear equation solver cannot achieve convergence after a few iterations, 2) sudden rate of change in displacement and 3) a failure mechanism is developed. However, this method has some limitations such as appropriate selection of constitutive model and geologic parameters, boundary conditions and defining a failure condition (Krahn 2007).

Another approach to solve a slope stability problem by finite element method is to compute finite element stresses of the geotechnical structure and to implement them inside a limit equilibrium frame work to analyze its stability. It is known as finite element stress-based approach (SLOPE/W 2010). So, in this approach the distribution of stresses in the ground are calculated by finite element analysis and then these stresses are used in a stability analysis. For the present case study involved in this thesis, this approach is followed. The software SIGMA/W is used for calculating the insitu stresses in the landslide and SLOPE/W is used for the slope stability analysis using the stresses calculated by SIGMA/W (SLOPE/W 2010). A brief description of both SIGMA/W and SLOPE/W softwares are discussed in further sections. The main advantages of this method are that there is no need to assume any interslice forces like in limit equilibrium methods, no convergence problems, computed ground stresses are close to reality and Soil-structure interaction effects are included etc.

2.2.3 Three dimensional slope stability analysis

2.2.3.1 Extension of mechanics of two dimensional (2D) limits equilibrium analysis to three dimensions (3D)

Though extensive studies are done in the area of two dimensional slope stability, slope failures that occur in nature have a three dimensional geometry. Hence understanding the variation of stability in a three dimensional view is important. A large number of studies were done on 3D slope stability analysis since late 1960's (Duncan 1996).The 3D analyses are commonly used in cases of a narrow failure surface, cuts or excavations, horizontal variation of ground water levels etc., (Hungr et al. 1989). The main difference of a three dimensional analysis from a two dimensional analysis is the consideration of spatial variation of the slope geometry and its geologic conditions. To render this purpose, the popular 2D limit equilibrium methods were extended into a third plane. The assumption made in 2D analysis i.e., dividing the sliding area into vertical slices has been extended as vertical columns and the method was commonly termed as the method of columns. Chen and Chameau (1983), Hungr (1987), Zhang (1988), Leshchinsky et al. (1985), Chang (2002), Hovland (1977), Baligh and Azzouz (1975) and others have contributed in this area of research. According to case study results published in Duncan (1996) for a particular slope the factor of safety for the most critical failure surface obtained from a 3D slope stability analysis is greater than the factor of safety obtained from 2D analyses. The factor of safety obtained from a 2D analysis is conservative and smaller than 3D (K. C. Zhang 2011). A 3D analysis considers the end effects of sliding surface, its lateral curvature and lateral non-homogeneity in its framework. All these factors are neglected in a 2D framework. Hence factor of safety obtained from 3D analysis is greater than the factor of safety obtained from 2D analysis (Hungr 1987). The Figure 2-2 represents forces acting on a typical column considered in a 3D slope stability analysis (method of columns). The forces acting on the column are similar to the forces acting on a single slice in 2D method of slices except that the forces are now considered in Z direction also. The forces shown here are based on direct extension of assumptions of Bishop's simplified method (1955).

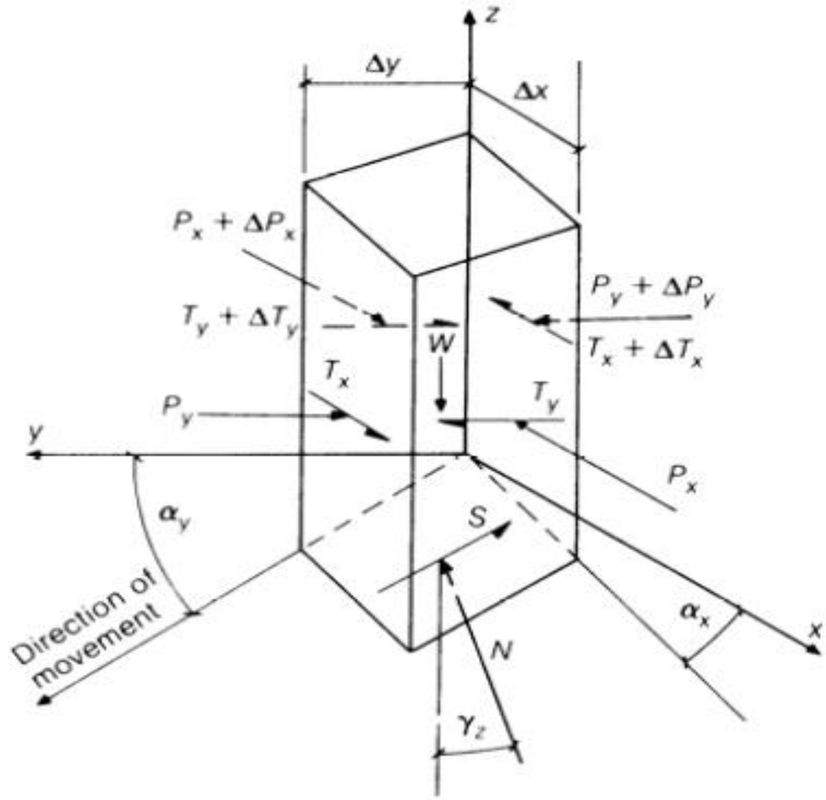

Figure 2-2: Forces acting on a single column (adopted from Hungr 1987).

Hungr (1987) extended the Bishop's simplified 2D method of analysis to a 3D method. The same assumption for the extension remained same as in 2D method – Vertical interslice forces are neglected. In the Figure 2-2 vertical inter column forces are neglected. Horizontal force equilibrium conditions both in *x* and *y* direction are neglected in this method and only the vertical and moment equilibrium conditions are considered and were sufficient for attaining the equilibrium. A detailed explanation and derivation of the 3D factor of safety equation are discussed in Hungr (1987). This method tends to be conservative in case of some non-rotational, asymmetric surface and in cases sliding surfaces with rapid mobilization internal strength (Hungr et al. 1989 & Hendron and Patton 1985). Similar work was done by Huang and Tsai (2000) but its factor of safety showed variation with variation in the sliding direction of the slope. Along with Bishop's simplified method, extensions have also been made to other 2D limit equilibrium methods such as Morgenstern-Price, Spencer's and Janbu's method. Janbu's method is reported as more conservative than the Bishop's method (Hungr 1987). An algorithm was developed by Hungr to computerize this method and was implemented in a microcomputer program called CLARA/W. Further description about CLARA/W can be found in section 2.5.3. The Spencer's method was extended from 2D to 3D by Chen and Chameau (1982). It assumes that the inter column forces have same inclination through the sliding area and also the inter column forces are assumed to be parallel to the base of the column. An extension of the Morgenstern and Price method into a 3D method was done by Cheng and Yip (2007) considering an asymmetrical sliding surface. A detailed formulation of this method using only force and moment equilibrium methods is shown in his paper. Bishop's and Janbu's method were also formulated by him in this paper based on the same assumptions. The advantage of these formulations is that it is highly applicable to nonsymmetrical surfaces and also considers transverse loads in 3D. In this thesis, for a 3D analysis of the Forest City landslide CLARA/W software formulated by Hungr (1987) is used.

2.3 Soil Strength and Stability Analysis Conditions

The most important part in slope stability analysis is selection of the shear strength properties of the soil. According to Lowe (1967) shear strength is the property which has greatest degree of uncertainty. An undrained analysis is a total stress analysis considering all the forces transmitted through interparticle contacts and water pressures. Undrained shear strength properties cohesion (c) and internal friction angle (ø) can be estimated by unconsolidated undrained triaxial test (UU), in situ tests and also consolidated drained test (CU) when considered with strength normalizing procedure (SHANSEP) (Ladd and Foott 1974). In the drained analysis only the forces transmitted through the particle contacts are considered. The drained shear strength properties are estimated by consolidated drained test (CU), direct shear test, SPT and CPT by using few correlations. However, in either of the cases external water pressure if any should be considered. Undrained and Drained properties are only related to the internal pore pressures in the soil.

For a slope stability analysis the decision regarding type of analysis is governed by the condition of the construction. Stability of a slope during and at the end of construction is analyzed by using drained or undrained strengths, based on the permeability of the soil. In case of long terms stability analysis which commonly reflects the cases of swelling and consolidation, shear strengths are expressed in terms of its effective stresses. In case of a staged construction which mainly reflects the case of consolidation, drained shear strength properties are considered. In case of drawn down condition, if the drawdown occurs suddenly then the soil cannot completely drain the water level in it, so an undrained analysis is performed (Duncan and Wright 2005). In the present study, an effective stress analysis is conducted in all conditions.

2.4 Constitutive Models for Finite Element Analysis

2.4.1 Modified cam clay model

Based on the available laboratory and field data, Modified Cam Clay model was selected to represent the soil in the finite element analysis. This section consists of brief information about the modeling basics, assumption, limitations of this model. A Cam-Clay model is based on critical state soil mechanics frame work using effective stress parameters. This model is formulated by Atkinson and Bransby (1978) and Britto and Gunn (1987).

The Figure 2-3 represents the variation of volume change with increase in pressure and elastic- hardening plastic curve respectively obtained from a one-dimensional consolidation test. This also depicts the analogy between the overconsolidation line in the volume-pressure relation to the initial linear elastic line in the stress-strain relation. When the overconsolidation line is rotated for 90^{o} it represents the elastic line in the stress-strain graph. Similar analogy is shown by the normal consolidation line and the hardening plastic line. Figure 2-4 graphically represents the definitions of the Cam-Clay parameters. The critical state line depicted in the graph is defined as a straight line joining (locus) all the critical points at each value of change in pressure.

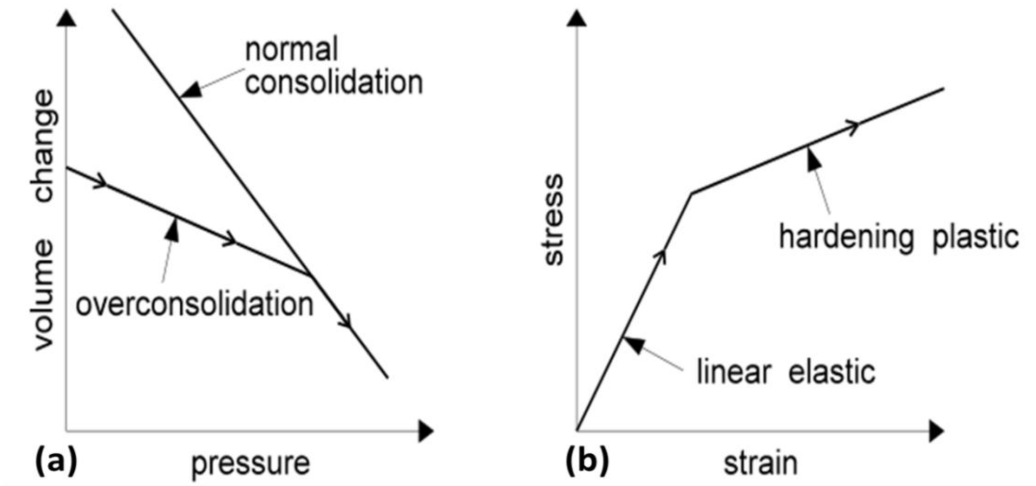

Figure 2-3: (a) Shows the variation of volume with pressure and (b) Shows the stress- strain relationship (SIGMA/W 2010).

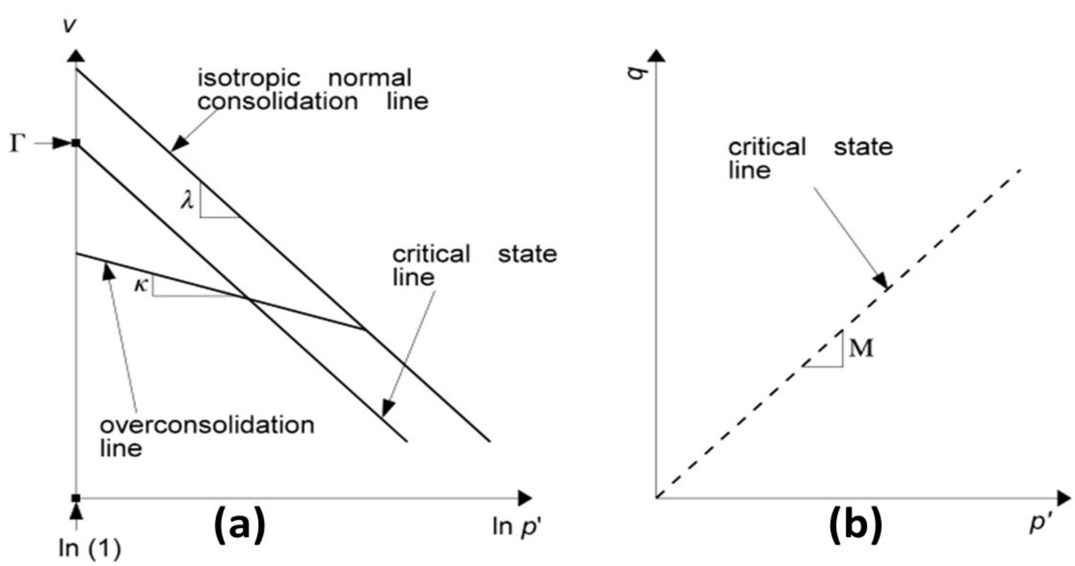

Figure 2-4: Graphical representation of the definition of the cam-clay model parameters (SIGMA/W 2010)

Where, p′ – Mean stress

q– Deviator stress

e – Initial void Ratio

The important parameters required for the analysis are as follows, M – Slope of the critical state line in p′-q plane, Γ – specific volume at critical state when p′ is 1.0, Κ – slope of the over consolidated line, λ – slope of the isotropic normal consolidation line and ν – Specific volume. These parameters can be directly obtained mainly by performing a one - dimensional consolidation test and a triaxial compression test. Mentioned below are few equations which can be used to estimate these parameters. For calculation of M, the effective friction angle $\emptyset'$ obtained from any other insitu or laboratory testing can be related as follows.

$$M = \frac{6sin\emptyset'}{3 - \sin\emptyset'} \qquad 2\text{-}4$$

$$\lambda = \mathrm{Cc} / 2.303 \qquad 2\text{-}5$$

$$\mathrm{K} = \mathrm{Cr} / 2.303 \qquad 2\text{-}6$$

To obtain the values of λ and Κ, the values of compression index (Cc) and re-compression index (C_r) obtained from the one-dimensional consolidation test through the void ratio (*e*) and pressure $\log_{10}$ (*p*) curve (*e* vs $\log_{10}$ (*p*) curve) are used as mentioned in Equations 2-5 and 2-6.

2.5 Simulation Softwares Used in the Analysis

In this report, GEOSTUDIO 2007 version is used for the purpose of two dimensional limit equilibrium and finite element analysis. In the GEOSTUDIO, SLOPE/W was used to perform the limit equilibrium analysis and both SIGMA/W and SLOPE/W were used for the finite element analysis. For 3D limit equilibrium analysis CLARA/W by O.Hungr (2001) was used.

2.5.1 SLOPE/W

SLOPE/W is a powerful commercially available tool for analyzing the stability of the slope. It is a component in the entire tool kit of GEOSTUDIO 2007. The first code for SLOPE/W was written by Fredlund and Krahn (1977). This software works on a limit equilibrium framework and includes methods such as the Morgenstern-Price, Spencer's method, Bishop's simplified method, Janbu's generalized method and Ordinary method slices etc. A finite element stress-based can also be solved using this software. Factor of safety for different shapes of slip surfaces – Circular, noncircular and wedged surfaces can be determined. The soil models such as Mohr-Coulomb model, anisotropic strength model, SHANSEP model, bilinear models etc. are available for modeling the material properties of the different layers of the soil. User can also easily define a particular slip surface in the slope. This feature is mainly advantageous in post slope failure analyses. The main advantage of this program is its ability to be coupled with other programs such as SIGMA/W, SEEP/W, VADOSE/W etc. By integrating these programs for a particular analysis a slope can be analyzed considering the aspects of the insitu stress – strain behavior, earthquake acceleration factors, seepage factors etc. Stability of reinforced slopes can be assessed using this software. Different types of reinforcements – anchors, geo-fabrics, soil nails, piles, sheet-piles etc. can be designed. Pseudostatic analysis and analysis for liquefaction stability can also be performed (SLOPE/W).

Illustrative examples are provided in the SLOPE/W manual (2007) for verification of the analyses using SLOPE/W program. These examples show a detailed comparison of the analysis results from SLOPE/W with solutions obtained from the Stability charts developed by Bishop and Morgenstern in 1960's, a comparison with published results and a comparison with theoretical calculations of earth pressures. The analyses results from SLOPE/W prove to be the same as the values obtained from the other methods, indicating that the results obtained using SLOPE/W program are reliable.

2.5.2 SIGMA/W

SIGMA/W is a finite element software product mainly used for determination of stress-deformations in earth structures. In addition, it is also used to model soil-structure interaction, staged constructions, consolidation analyses etc. SIGMA/W is a component of GEOSTUDIO 2007. It was designed in a way to analyze both simple and complex problems because of its available options for different analyses. Performing both simple linear elastic analysis and complex non-linear plastic analysis is possible using this software (SIGMA/W 2010). The discretization into finite elements is done by creating a mesh over the cross section of geotechnical structure. Mesh generation is an automated process in SIGMA/W. A choice for the shape of mesh elements – Quadrilateral, triangular rectangular and mixed shapes are also available. Density of the mesh can be varied as required. Boundary conditions must also be defined for a SIGMA/W model. The available boundary conditions are Fixed X, Fixed Y and Fixed X/Y. Generally, for in calculating stresses for a slope stability problem the boundary for the base is selected as Fixed X/Y, as the displacement in both X and Y directions is not allowed at the base. The significance of SIGMA/W in slope stability analysis is that it can model the stress deformations along with the pore water pressures that arise due to stress change. Also, it readily provides a graphical representation of variation of the stress strain values with variation in loading, time, ground water level etc. In this thesis, for a finite element analysis of the Forest City landslide this program is coupled with SLOPE/W to obtain the safety factor calculated considering the stress – strain relationship of each soil layer.

Illustrative examples are included in the SIGMA/W manual (2007) for verification of its formulation. These examples show that the values calculated using SIGMA/W program match with the values obtained from hand calculations and published values.

2.5.3 CLARA/W

CLARA/W is a 3D slope stability analysis program following limit equilibrium frame work. The algorithm for this program was written by Hungr (1987). A 2D analysis can also be performed in this software. The complete area is defined by required number of cross sections covering the entire extent of landslide. The profiles of these cross sections are interpolated to form a complete three dimensional slope. Figure 2-5 represents a typical 3D sliding surface obtained using CLARA/W. The mesh represents the plan view of the column assembly.

Different shapes for the slip surfaces such as ellipsoidal, circular and wedge are available. This program also provides the facility of importing the cross section definitions from various formats. In this program, same soil properties have to be considered for each cross section. Variation can only be given in the definition of layers and the piezometric layer for each section. The analysis is performed using the method of columns, as discussed earlier. In the output of the analysis few checks regarding the number of active columns, percentage of unbalanced forces, number of rows and columns used for analysis etc. are always necessary to validate the result obtained.

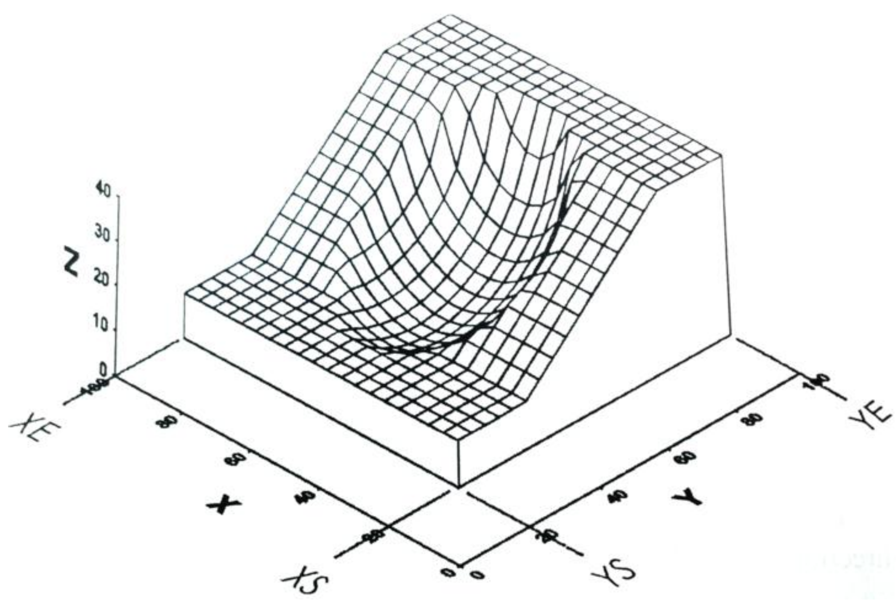

Figure 2-5: Typical CLARA/W representation of plan view of a 3D sliding surface (adopted from CLARA/W manual 2001)

CLARA/W is based on extension of four limit equilibrium methods into 3D. The first is Bishop's Simplified method based on formulations given by Hungr (1987) and considering the limitations specified by Fredlund and Krahn (1977). Second, Janbu Simplified method is extended in same lines as the 2D method. Next, Spencer's and Morgenstern-Price were also proposed by Lam and Fredlund (1993). All the four methods give similar results for rotational sliding surface geometries (CLARA/W manual 2001). The Janbu's method usually yields lower factor of safety values compared to the other methods. Also, in some cases Spencer and Morgenstern-Price methods fail to converge. The Bishop's Simplified method may be inaccurate in case of large horizontal external loads. But in CLARA/W the Bishop's method has the facility of identifying the presence of lateral imbalance and can be balanced by a method described by Hungr (1997). Morgenstern-Price and Spencer's method do not have this facility in CLARA/W. In this thesis, the stability analysis of the Forest City landslide was conducted using Bishop's Simplified method, Morgenstern-Price and Spencer's method. But as there was no convergence obtained by the analysis performed by the Morgenstern-Price and Spencer's methods these methods were not reported in the analysis section. Trials to make them converge were also conducted by decreasing the mesh density but then the methods were failing for the necessary check of having the number of active columns analyzed to be greater than 1000.

2.6 Background of Pierre Shale:

Pierre Shale is marine clay from the Cretaceous age, 60 to 80 million years ago. This weakly cemented shale was deposited in and around the Cretaceous Epeiric Sea located in the central part of United States and Canada. It mainly consists of clay shale material along with layers of bentonite, smectite, clay stone, silt and sand stone. This shale is strong at depth but when exposed to the atmosphere it gets weakened by weathering and desiccation. Transgression and regression of the sea, volcanic ash deposition caused non-homogeneity of materials in the Pierre Shale. This non-homogeneity is noticed in the form of presence of varying thickness of bentonite layers from several feet to few millimeters thick (Tourtelot 1962).

The Missouri River trench located South Dakota divides the state into east and west halves. It was formed in Sangamon time during Pleistocene glaciations about 135,000 to 70,000 years ago as a result of melting of glaciers (Flint 1955). The melting glaciers on the east side of the river and due to erosion of the clay shale materials on the west side caused unloading of the nearby surface materials. Unloading and weathering of this shale produced heavily overconsolidated clay that readily fails due to minor disturbances. The weathering in this material was noticed in the form of formation of tension cracks, fractures and fissures triggering numerous landslides in this area. Early landslides were reported to be observed in Late Wisconsin times about 25,000 to 10,000 years ago (Crandell 1958). These landslides overtime filled the Missouri River trench with sediments of 60 to 100 ft. of alluvium. These alluvium sediments consist of poorly sorted gravel, silt and clay underlined by the Pierre Shale formation. Hence it is evident from the geologic history that this valley is prone to landslide activity and consists of fractures from ancient landslides. Based on this fact, to understand the behavior of Pierre shale understanding its residual strength behavior is extremely important. The construction of main stem dams along the Missouri River trench has significantly influenced many researchers to understand the residual strength behavior of the Pierre shale. A summary of results obtained for residual strength of Pierre Shale in various locations in and around the Missouri River trench are presented in Table 2-2.

The values from the Law Engineering Company report (1976), Dames & Moore, Corps of engineers from SDDOT reported in the above table are mentioned in the Task I-B report of the Grenier and Woodward Consultants (1991) on the Forest city landslide. The Table 2-2 gives a brief idea on the range of the residual shear strength values previously obtained by other researchers. According to values reported in the above table it is observed that the unit weight of the sheared shale ranges from 115 to 130 pcf. From Table 2, it can be observed that the residual frictional angle obtained varies from 3.1^{o} to 8.1^{o}. Further in this thesis, analysis of the Forest City landslide is shown in two dimensions and three dimensions to observe the variation of the residual strength of the Pierre Shale layer.

Table 2-2: Reported Residual friction angles for Pierre Shale (Schaefer 2002) and Wood ward Clyde company (1991)

Reference	Location	Description	C (pcf)	Ø'	Test Performed
Fleming, Spencer & Banks	Oahe Dam	Shale remolded	0	Ranges from 5.1^o to 7.4^o	RDS precut
Bump	Forest city	Shale remolded	0	6.4^o to 8.0^o	RDS precut
		Bentonite layer	0	2.8^o to 4^o	RDS precut
Townsend & Gilbert	Oahe Dam	Shale remolded	0	Ranges 3.1^o to 3.3^o	Rts, RDS precut and Rgs precut
Stark & Eid	Reliance, SD	Shale	0	6.5^o	Rgs
	Oahe Dam	Firm Shale	0	7.4^o	Rgs
	Oahe Dam	Bentonite Shale	0	6.0^o	Rgs
USACE	Oahe Dam	Weathered shale remolded	0	3.8^o	Rts
Schaefer & Lones	Forest City	Failure Zone – distilled water	0	7.2^o	RDS
	Forest City	Failure Zone – distilled water	0	8.1^o	RDS
Law Engineering testing Company	Forest City	Sheared Shale	135	7^o	-
Dames & Moore	Forest City Landslide	Sheared Shale	0 – 300	3.2^o	-
Corps of Engineers	Oahe Dam	Sheared Shale	0 – 300	8.5^o	-
SDDOT	Forest City Landslide	Sheared Shale	0 - 200	5^o	-

RDS precut = Reversal Direct Shear test (The undisturbed samples are precut horizontally to form the failure surface);

Rgs = Ring shear test;

Rts = Rotational shear test.

CHAPTER 3. CASE STUDY: FOREST CITY LANDSLIDE

3.1 Introduction

This section consists of a detailed description of the landslide. The Forest City landslide was selected for study due to the availability of extensive geotechnical data for the analysis. The geologic history of the landslide area and its stratigraphy are discussed here. A brief discussion on back analysis procedure for determining the residual shear strength properties of the failure causing layer at the time of failure is included in the discussion. Also, cross sections used for the 2D and 3D analyses are discussed in this section. Detailed figures showing the plan view of the cross sections used are included in the appendix.

3.2 Forest City Landslide

A large landslide was reactivated in early 1960's on the banks of the Oahe Reservoir, South Dakota where the U. S Highway 212 crosses the Oahe Reservoir via Forest City Bridge. This site is located about 50 miles north of Pierre, South Dakota. The location of the site is shown in the Figure 3-1. The Forest City Bridge was funded by the U.S Corps of Engineers and was built as a replacement to a bridge crossing the Missouri river approximately seven miles upstream. There was a requirement for replacement of the bridge because impoundment waters from the construction of the Oahe dam would cease the usage of the bridge. Oahe dam was being constructed about 50 miles downstream from the Forest City Bridge. The construction of the dam and bridge were completely nearly at the same period in 1958. The filling of the reservoir started in 1958. The normal operating pool level of 1585 ft was reached by 1962 and it reached 1600 ft by 1968. The highest reservoir level was marked to be at 1620 ft (Grenier and Woodward Consultants1991).

Unknowingly the bridge was built on the toe of an ancient landslide located at the southern approach of the bridge. The bridge is shown in Figure 3-2. The rise in the elevation of the pool level reactivated the landslide (Schaefer 2002). Movements and distress were observed the 5000 ft approach roadway near the southern embankment (Figure 3-2).

Cracking of the pavement was found about 500ft from the end of the bridge. Many localized failures, surface tension cracks were observed near the southern embankment indicating that the entire mass was moving towards the bridge. This landslide was classified as a progressive landslide (Grenier and Woodward Consultants1991). However, the northern approach of the bridge was reported to be located on a relatively stable ground. The movements due to the landslide threatened the structural integrity of the bridge. The bridge at that time was an important commercial transportation link connecting the Cheyenne River Indian Reservation and Western cattle agriculture areas with eastern markets and commerce centers. The temporary or permanent closure of this bridge could have caused approximately 85 mile of detour (SDDOT 1981).

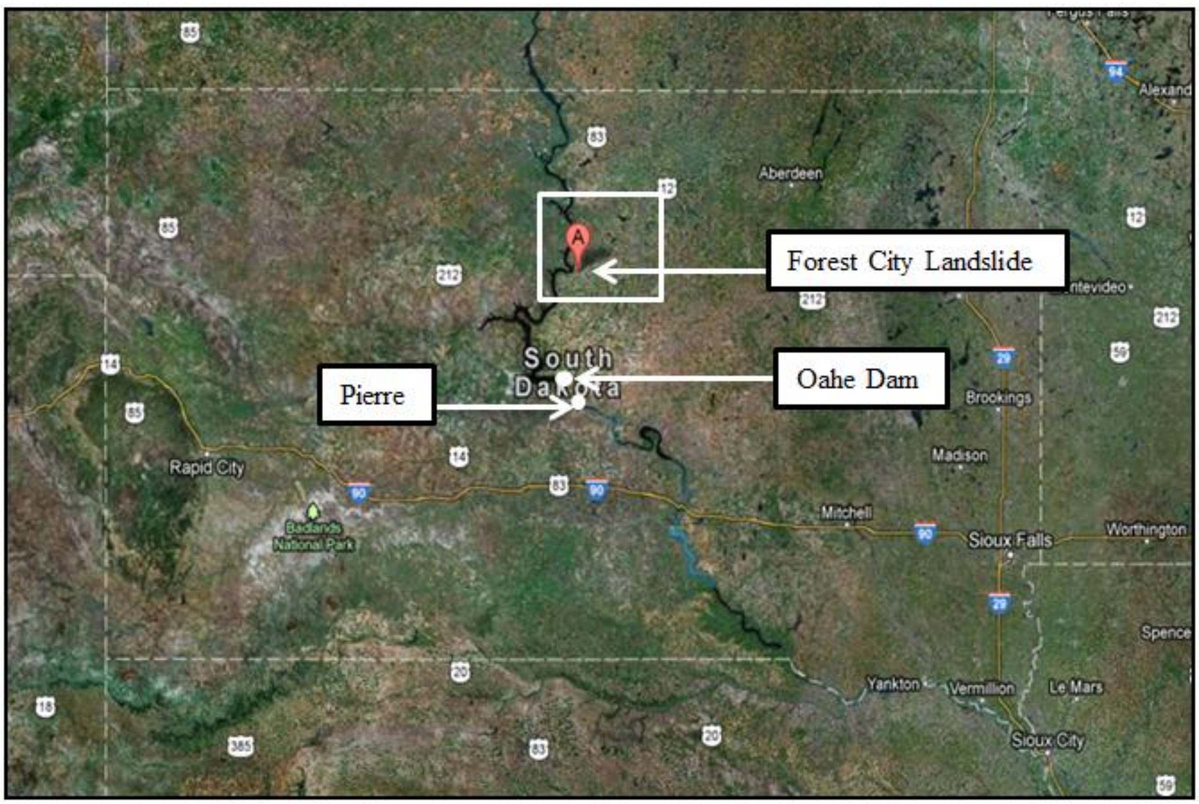

Figure 3-1: Location of Forest City Landslide and Oahe Dam (Google earth view)

Extensive testing's of the movements of the bridge was started in 1972. Continuous movements were observed until 1980 and remedial investigations were started in 1988. The stabilization measures included installation of stone columns in the abutment, unloading of the driving force by cutting and installation of shear pins (Grenier and Woodward Consultants1991).

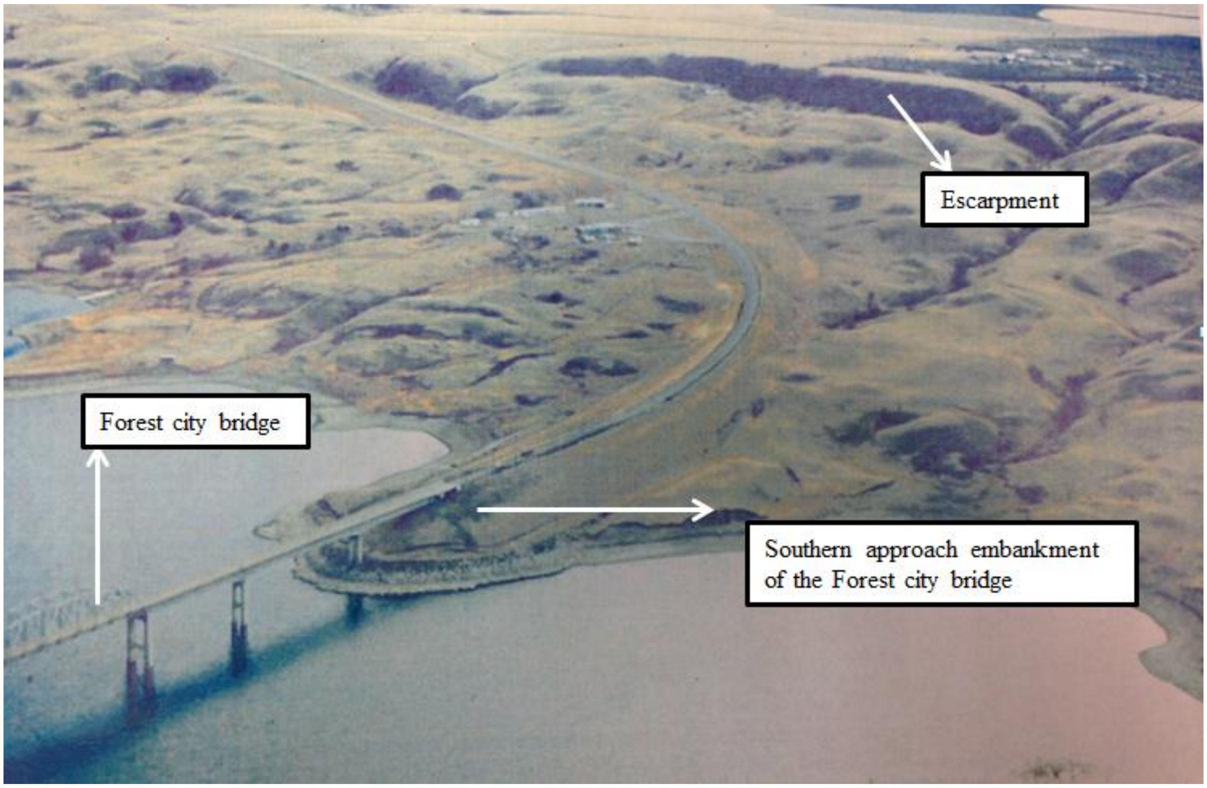

Figure 3-2: Aerial photograph of the Forest City Landslide (adopted from Grenier and Woodward Consultants1991).

A detailed report on the geotechnical investigations consisting of the borehole data, laboratory and field investigations are provided by the geotechnical testing firms – Law Engineering Testing Company (1976), Woodward – Clyde Consultants (1991), Aaron-Swan Associates and Grenier Ins (1991). These consulting firms were contracted by the South Dakota Department of Transportation for performing the required geotechnical investigation. Herein, the geotechnical data from these reports is directly adopted for the analysis purpose.

3.3 Geologic History of the Landslide

The site is located in the hilly area of the Missouri River trenches. It is situated on the west edge area of the Glaciated Missouri Plateau which is underlain by Pierre shale, bentonite marine shale of the Cretaceous age. A description of the geologic history of the Pierre shale is included in Section 2.6. The blockade of the Cheyenne River and the other northeastward flowing rivers by the glacier created the existing Missouri river channel. The river trench formation lead to the removal of lateral support and a series of complex slump block failures were formed along the shoreline. The large longitudinal block failures along the shoreline also progressed upslope and thereby caused slump blocks moving towards the channel (Grenier and Woodward Consultants1991).Landslide activity is not uncommon in this area (Schaefer 2002). The surficial material as reported by the borehole data from the Corps of Engineers (1950), Law of Engineering Testing Company (1976) and the South Dakota DOT (Forest City landslide, Geotechnical Report, 1981) consists of glacial till materials with concentrations of gravel and boulders. A fill type material is also found along the abutment of the bridge which was placed at the toe of the bridge during the construction of the abutment of the bridge. A 5 to 10 ft thick layer of loess material is found near by the escarpment. All these materials are underlain by layers of weathered and firm Pierre shale. Extensive monitoring of the site for shear failure surfaces revealed that the failure surface is at or just above the contact of the weathered shale and fresh shale layer.

To perform a stability analysis for the landslide a reasonable area representing the landslide characteristics has been selected by the SDDOT. The area ranges from the toe of the south end of the bridge abutment to the escarpment at the upper portion of the slide. The area beyond the escarpment is assumed to be stable (Forest city landslide, Geotechnical Report, SDDOT, 1981). The entire area of the landslide consists of numerous number of tension cracks filled with water. Although the presence of tension cracks and slide scars make the area to look like a series of sliding blocks, the central corridor area from the shore to the escarpment are considered as a single unit for the purpose of the analysis. The entire sliding mass tends to move northward away from the escarpment. The elevation of the central corridor ranges from 1550 ft to 1800 ft. Figure 3-3 shows the idealized section of the central corridor of the landslide modeled using SLOPE/W.

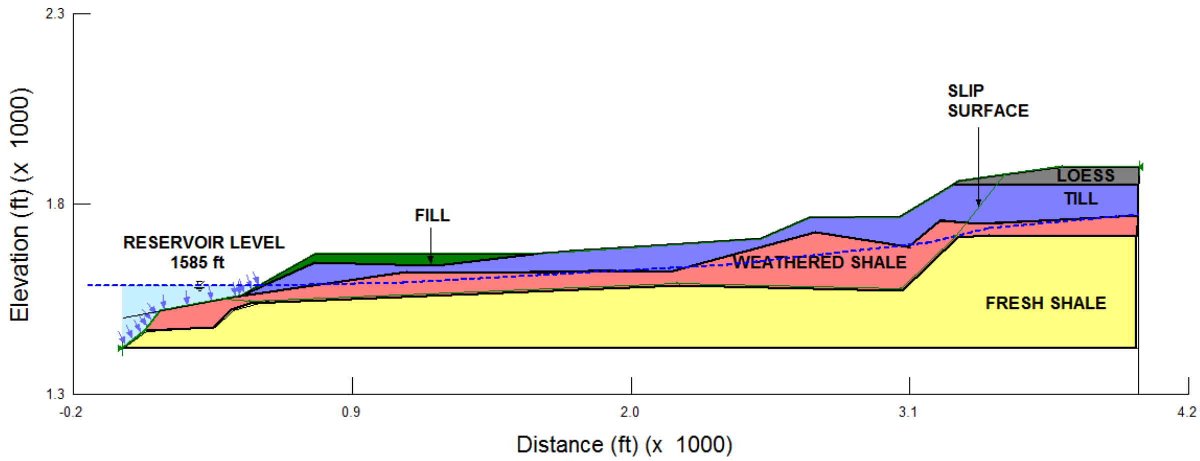

Figure 3-3: SLOPE/W model of the Central corridor area of the Forest City landslide.

3.4 Geologic Sections and Parameters of the Area

For a 2D analysis the geologic section AA' as shown in Figure A-1 in the appendix is considered. This section is considered to be representative of the overall landslide in 2D and ranges from the toe of the southern embankment to the main escarpment (SDDOT 1991). The area of the section is 4000 ft x 1900 ft. The stratigraphy of the section and the ground water table elevation are obtained using the borehole log data and the inclinometer data reported by the Woodward Clyde Consultants (1991). The geologic parameters assigned for each layer are shown in Table 3-1. The required geologic parameters are obtained from the Task I-A, Review of Available Data (1989) and Task I-B, Preliminary Evaluation of Landslide Stabilization (1989) reports prepared by Greiner, Inc. for the SDDOT. In case of the 3D analysis, the representative model is prepared by considering required number of cross sections over the entire landslide area. Each cross section is defined in the similar way as done in 2D analysis. All the soil layers and the piezometric line have to be defined at each cross section. The 3D profile is prepared by the software by interpolating among the layers in each defined cross section. CLARA/W is used for the 3D analysis. A description about CLARA/W is in section 2.5.3.

Table 3-1: Summary of the geologic parameters (Grenier and Woodward Consultants1991)

S.No	Soil Layers	Unit weight, γ_m (pcf)	Cohesion, c (psf)	Ø° (degree)
1.	Fill	140	500	17
2.	Loess	130	500	19.3
3.	Till	130	500	19.3
4.	Weathered Shale	115	0	6 - 8
5.	Fresh Shale	120	1000	15

3.5 Back Analysis for Residual Shear Strength Properties

The slope instability in this case is due to reactivation of an old landslide (SDDOT 1991). Hence, the behavior of the slide is governed by the residual strength properties. Residual properties can be obtained either by conducting the traditional back analysis or by laboratory testing. The reliability of the value obtained from back analysis is proportional to the confidence with which the pore water pressure and the location of the slip surface in a slope are known (Bromhead and Dixon1986). Because of extensive availability of instrumentation data in this case the defined slip surface and the ground water elevations reported are considered to be reliable (Schaefer 2002). In this study, a back analysis is performed to determine the residual friction angle ($\emptyset_r$) along the slip surface. Location of the slip surface is shown in Figure 3-3. The cohesion value is set to zero and the frictional angle value is varied to obtain a factor of safety of unity (1.0) (Duncan and Wright 2005). According to Banks (1971), the range of residual friction angle of Pierre shale is expected to be between 5° and 8°. These values are obtained from direct shear tests in the laboratory.

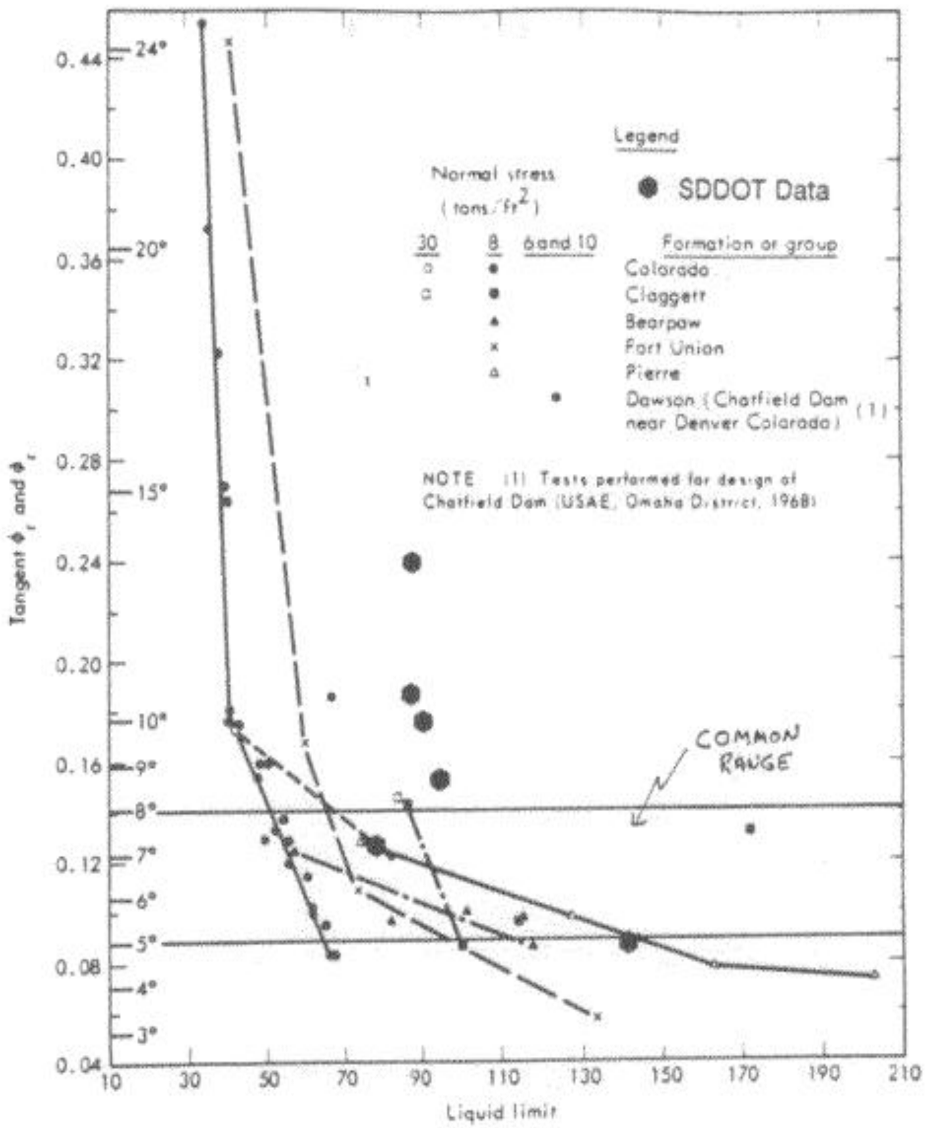

Figure 3-4: A summary of residual friction angle obtained from direct shear strength test vs. liquid limit for various marine shales of the northwestern United States (obtained from Banks, 1971) (Grenier and Woodward Consultants1991).

CHAPTER 4. ANALYSIS

4.1 Introduction

This section presently the results of back analyses of the Forest City landslide to determine its residual shear strength properties (c', $\text{Ø}_r'$) along the critical failure surface at the time of its failure. Based on the findings by the South Dakota Department of Transportation, the rise in reservoir levels following the completion of the Oahe dam played a significant role in the reactivation the landslide along with numerous other factors (Grenier and Woodward Consultants1991).To understand the effects of the rise in the water level on the residual friction angle of the failure causing layer (weathered shale), a parametric study was conducted to determine the effect of rise in the reservoir level on the residual friction angle. The analyses were performed using 2D limit equilibrium methods, 2D finite element method and 3D limit equilibrium method. Morgenstern – Price, Spencer and Bishop's Simplified methods are considered in case of 2D limit equilibrium method. Only Bishop's Simplified method is considered in case of 3D limit equilibrium analysis because of convergence problems with Morgenstern-Price and Spencer's methods. The reservoir level just after the completion of the construction of the Oahe dam was reported to be 1435 ft and reached its normal operating level of 1585 ft by 1962 and later raised to 1600 ft by 1972. The highest reservoir level was reported to be 1620 ft. Hence the reservoir levels of 1585 ft, 1600 ft and 1620 ft were selected to perform the analyses. The ground water level has been varied proportionally with the reservoir level.

4.2 Two dimensional analyses using Limit Equilibrium Method

The geologic cross section selected for a 2D analysis is discussed in Section 3.4. A 2D view analyzes the vertical cross section with a unit width. The plan view of the section is included as Figure A-1 in the appendix. The analysis is performed in Morgenstern – Price, Spencer, Bishop's Simplified methods. The software used for this purpose is SLOPE/W from GEOSTUDIO version 2007. Description of the software is included in Section 2.5.1.

First, the reservoir level considered to be 1585 ft. The ground profile of the slope consists of five different soil layers – the Fill, Loess, Till, Weathered shale and the fresh shale as shown in Figure 4-1 and the geologic parameters considered are as discussed in Table 3-1. In Figure 4-1 the blue dotted line represents the water level and the slip surface is shown starting from the layer of loess and further passing along the border of contact of the weathered shale and the firm shale. All the soil layers are modeled using the Mohr – Coulomb's model. As there was no information about the variation of geologic parameters in the vertical direction, a linear-elastic behavior is considered. This soil model is kept constant for analyses performed at all the three water elevations.

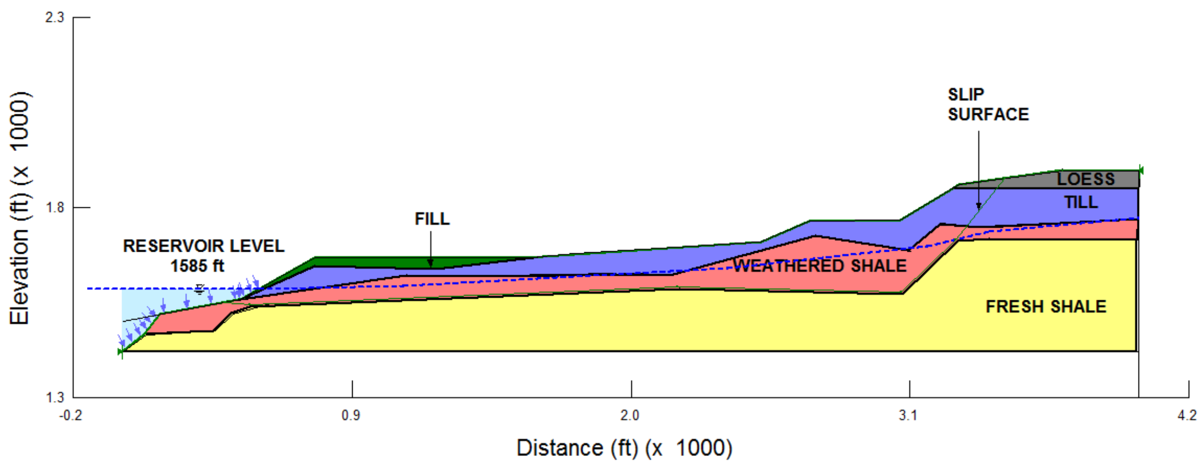

Figure 4-1: Slope/W representation of the slope at reservoir elevation of 1585 ft

Figure 4-2 represents the analyzed cross section of the slope at a reservoir level of 1585 ft. The green colour shaded area represents the sliding surface area The Figure 4-2 represents the analysis using the Morgenstern – Price method. The residual friction angle obtained by this analysis is 6.21°. The factors of safety values obtained by the other limit equilibrium methods at this particular friction angle value are tabulated in Table 4-1. This table shows the variation in the limit equilibrium methods for the Forest City landslide under same geologic conditions.

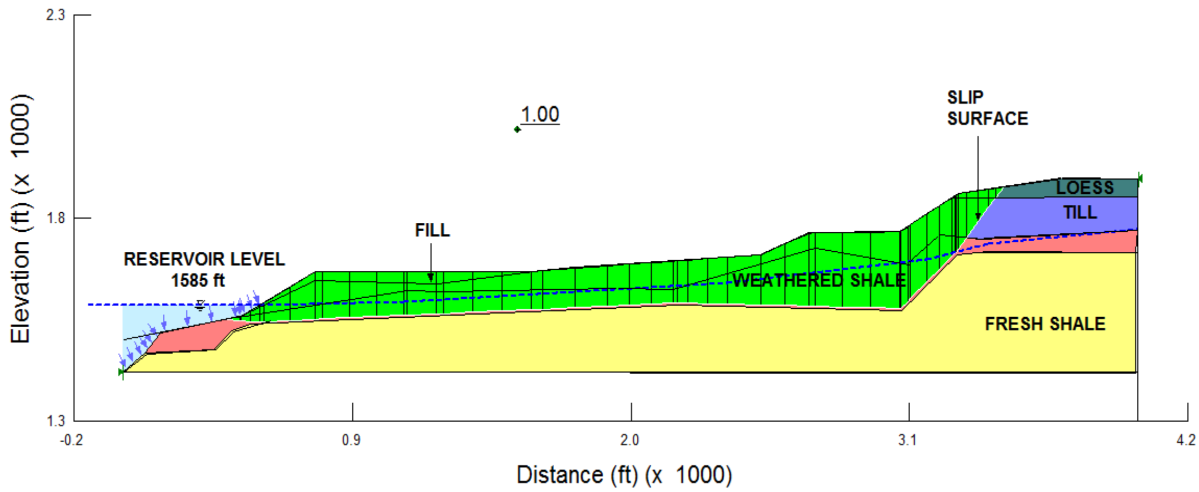

Figure 4-2: Slip surface at reservoir level of 1585 ft

Table 4-1: Summary of Factor of safety values for 2D LEM analysis at reservoir level of 1585 ft

Limit Equilibrium Method	**Reservoir Level (ft)**	**Residual Friction Angle ($Ø_r'$)**	**Factor of Safety (FOS)**
Morgenstern - Price	1585	6.21°	1.000
Spencer			1.012
Bishop's Simplified Method			1.181

Second, the analysis is performed at the reservoir level of 1600 ft. The Figure 4-3 represents the section. The cross section is same as the section used for the analysis at reservoir level of 1585 ft. The only variation is the reservoir level. Also, the ground water level is varied proportionally with the reservoir level.

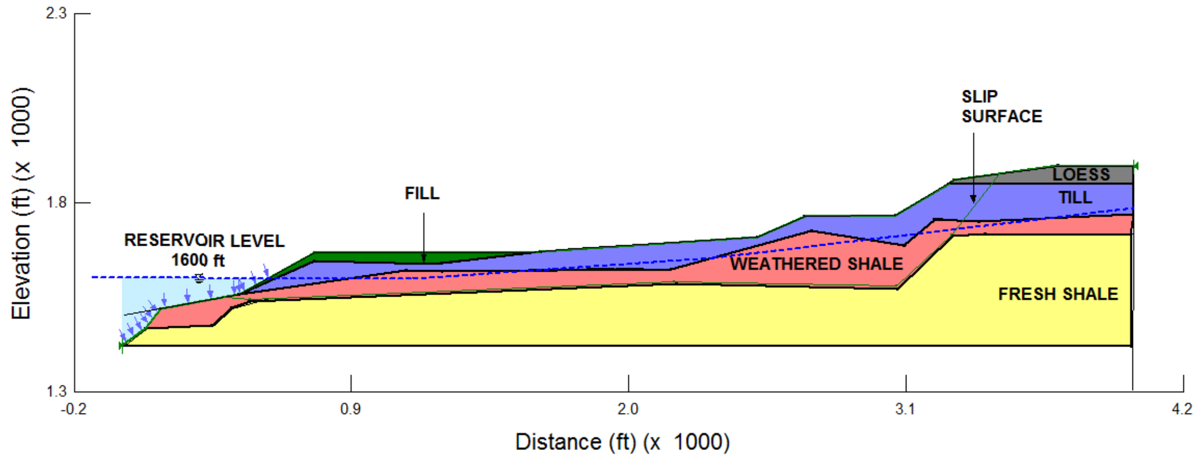

Figure 4-3: Slope/W representation of the slope at reservoir elevation of 1600 ft

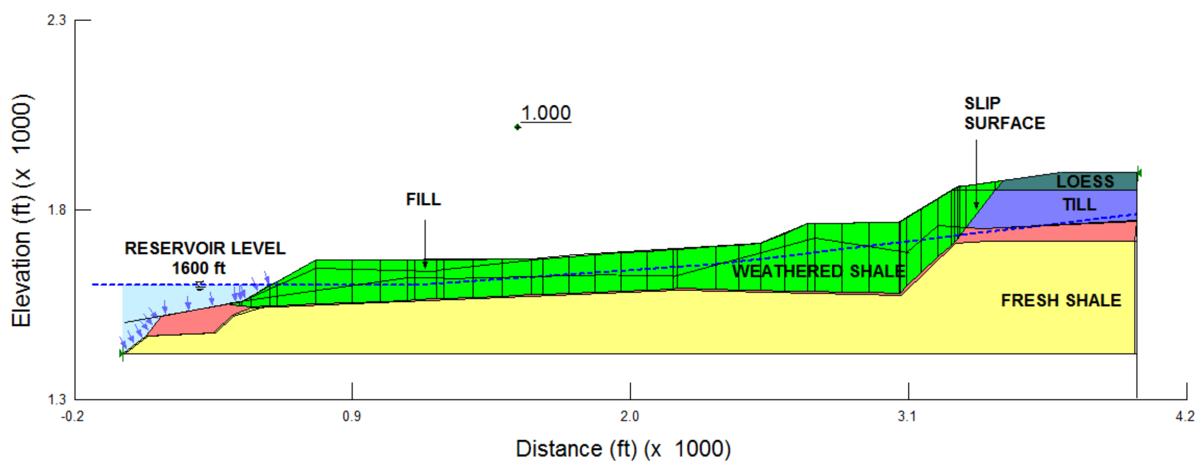

Figure 4-4: Slip surface at reservoir level of 1600 ft

Figure 4-3 represents the analyzed cross section of the slope at a reservoir level of 1600 ft. The green colour shaded area represents the sliding surface area The Figure 4-3and Figure 4-2 represents the analysis using the Morgenstern – Price method. The residual friction angle obtained by this analysis is 6.61°. The factors of safety values obtained by the other limit equilibrium methods at this particular friction angle value are tabulated in Table 4-2. This table shows the variation in the limit equilibrium methods for the Forest City landslide under same geologic conditions.

Table 4-2: Summary of Factor of safety values for 2D LEM analysis at reservoir level of 1600 ft

Limit Equilibrium Method	Reservoir Level (ft)	Residual Friction Angle ($Ø_r^{'}$)	Factor of Safety (FOS)
Morgenstern - Price	1600	6.61°	1.000
Spencer			1.011
Bishop's Simplified Method			1.187

Third, the analysis is performed at the reservoir level of 1620 ft, the highest reservoir level. The Figure 4-5 represents this section. The cross section is same as the section used for the analysis at reservoir levels of 1585 ft and 1600ft. The only variation is the reservoir level.

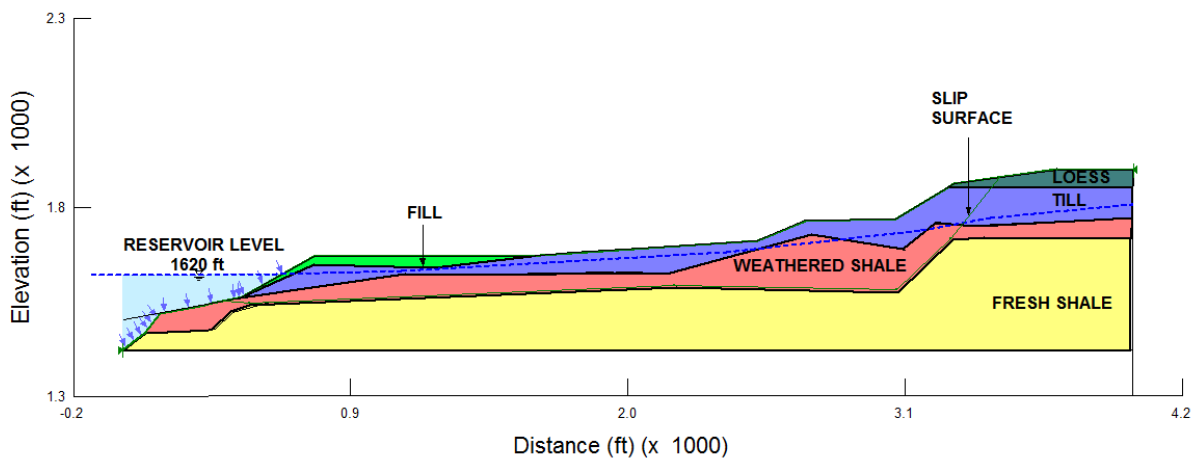

Figure 4-5: Slope/W representation of the slope at reservoir elevation of 1620 ft

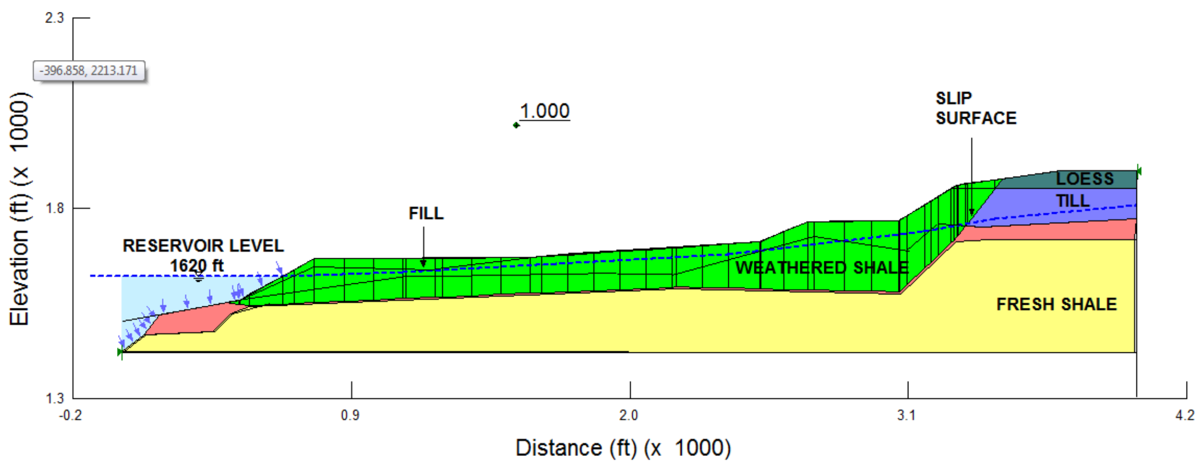

Figure 4-6: Slip surface at reservoir level of 1620 ft

Figure 4-6 represents the analyzed cross section of the slope at a reservoir level of 1620 ft. The green colour shaded area represents the sliding surface area. The Figure 4-6 represents the analysis using the Morgenstern – Price method. The residual friction angle obtained by this analysis is 7.4°. The factors of safety values obtained by the other limit equilibrium methods at this particular friction angle value are tabulated in Table 4-3. This table shows the variation in the limit equilibrium methods for the Forest City landslide under the same geologic conditions.

Table 4-3: Summary of Factor of safety values for 2D LEM analysis at reservoir level of 1620 ft

Limit Equilibrium Method	**Reservoir Level (ft)**	**Residual Friction Angle ($ø_r^{'}$)**	**Factor of Safety (FOS)**
Morgenstern - Price	1620	7.4°	1.000
Spencer			1.006
Bishop's Simplified Method			1.186

4.3 Two Dimensional Analyses for Residual Shear Strength Properties Using Finite Element Method:

Finite element analysis is done by including the stress-strain relationship of the soil layers in the traditional stability analysis. So to perform a slope stability analysis using the finite element method in GEOSTUDIO both SIGMA/W and SLOPE/W softwares are used. SIGMA/W is used for calculating the stress values and SLOPE/W is used for conducting the stability analysis as done in limit equilibrium methods. Traditionally the slope stability analysis using finite element method was done by the strength reduction method. In the strength reduction method an elastic-plastic analysis is performed by equally reducing the shear strength properties (c', $\text{Ø}_r'$) of all the soil layers by a particular factor i.e., weakening the soil artificially, until the slope fails (Griffiths and Lane 1999). This is done by using the stress redistribution type of analysis specified in SIGMA/W. However, the strength reduction method has some limitations as discussed by (Krahn 2007). Hence, it is a more preferred method to calculate the stresses using SIGMA/W and implement those stresses in a SLOPE/W model and perform the conventional trial slip surfaces stability analysis (SLOPE/W 2010). This method is known as the finite element stress based method and a detailed explanation of the method is included in SLOPE/W 2010 manual.

In the present case study, a Modified Cam Clay model is considered to represent the soil model. The Modified Cam Clay model is discussed in Section 2.4.1. The procedure to perform the stability analysis specified in the SIGMA/W manual is followed. Initially, the in situ stress values are calculated for the cross section specified in Figure 4-7 using insitu type model in SIGMA/W. Next, the cross section with the stresses calculated is analyzed traditionally by SLOPE/W with the defined slip surface. The required factor of safety is obtained (Unity value in this case). Figure 4-8 represents the back analyzed model solved by SLOPE/W using initial stresses calculated from SIGMA/W. The geological parameters, elevation of the reservoir level and the slip surface are same those considered for the 2D limit equilibrium analysis. The analysis is performed at three different reservoir levels – 1585 ft, 1600 ft and 1620 ft. The fixed X/Y boundaries conditions are implemented for all the finite element models and a mesh element size of 400 ft. The shape of the mesh is defined by quads and triangles.

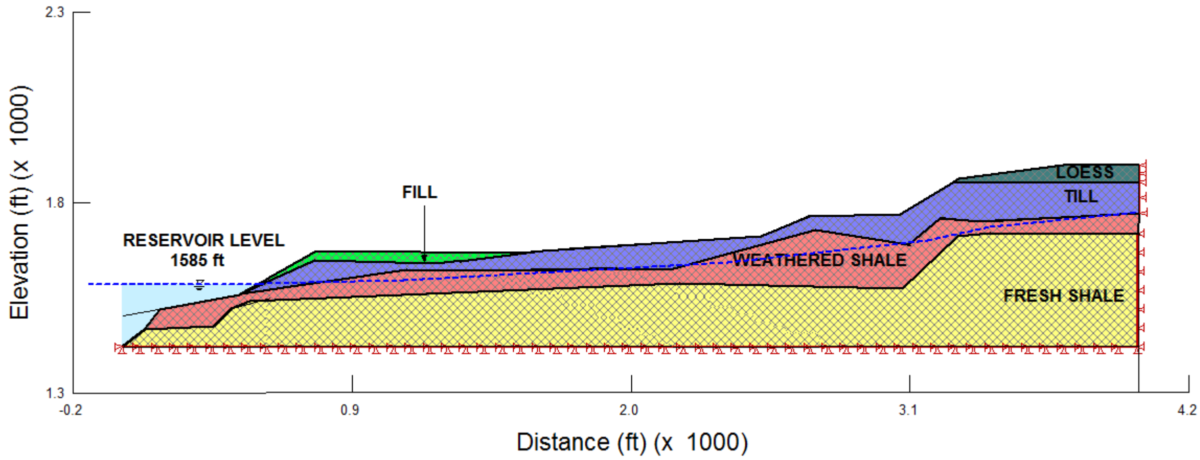

Figure 4-7: Cross section at reservoir level of 1585 ft with finite element mesh for computing the insitu stresses

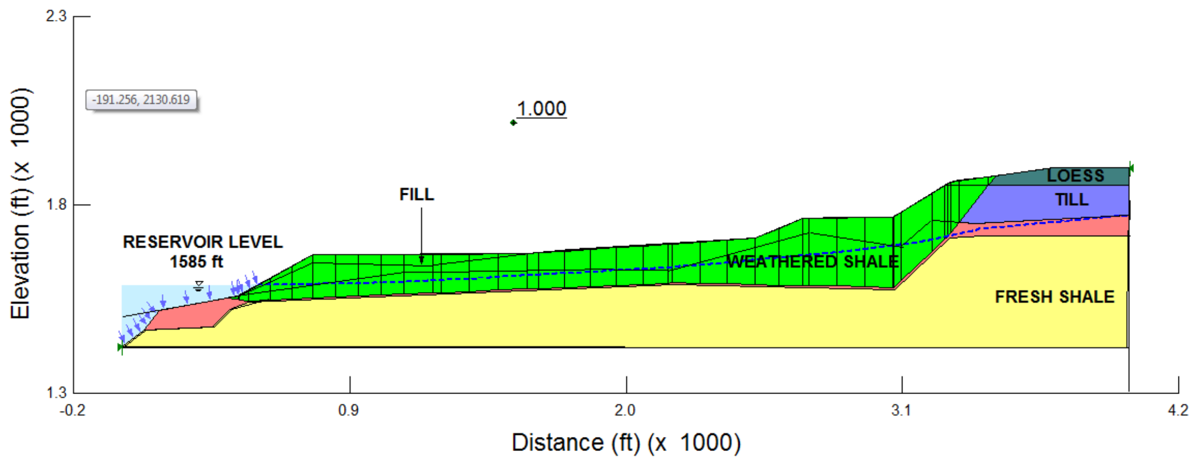

Figure 4-8: Back analyzed section in SLOPE/W showing the obtained factor of safety

Figure 4-7 and Figure 4-8 represent the finite element analysis performed at the reservoir level of 1585 ft. The residual friction angle value obtained at this reservoir elevation is 6.64°.

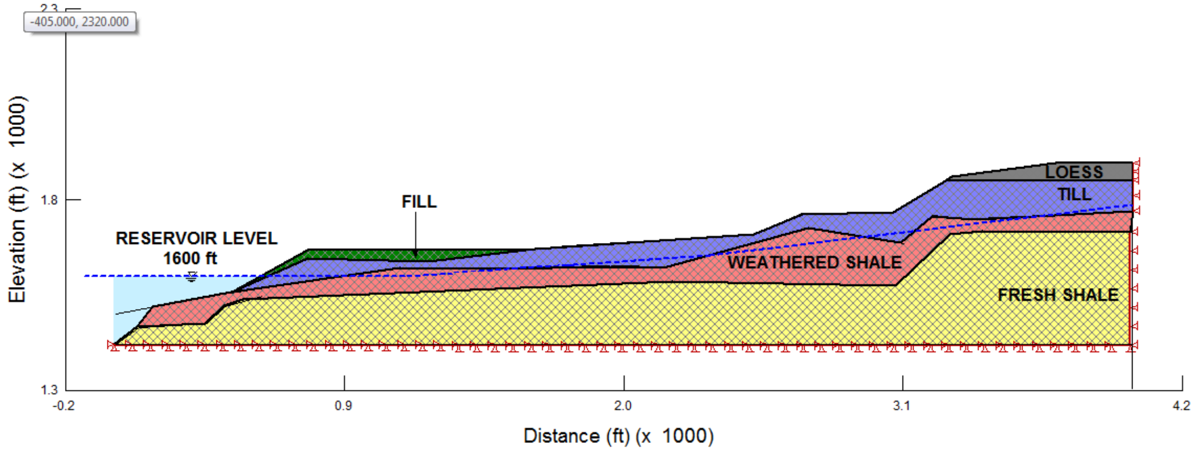

Figure 4-9: Cross section at reservoir level of 1600 ft with finite element mesh for computing the insitu stresses

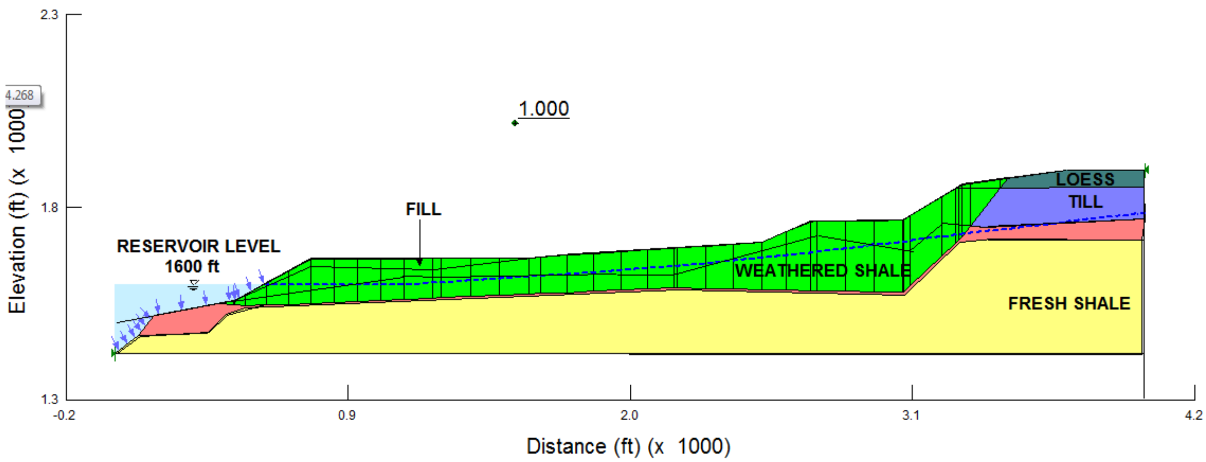

Figure 4-10: Back analyzed section in SLOPE/W showing the obtained factor of safety

Figure 4-9 and Figure 4-10 represent the finite element analysis at the reservoir level of 1600 ft. The residual friction angle obtained is 7.165°.

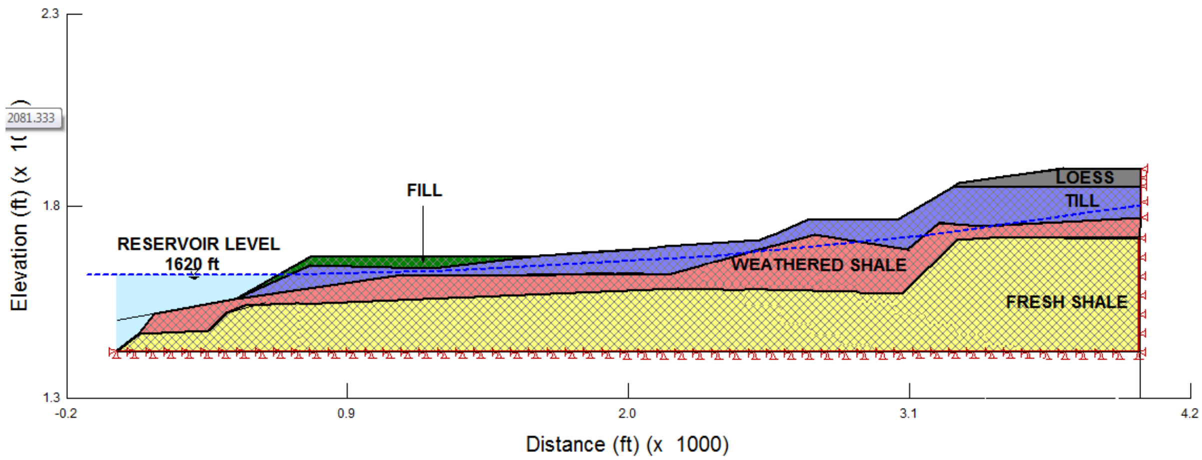

Figure 4-11: Cross section at reservoir level of 1620 ft with finite element mesh for computing the insitu stresses

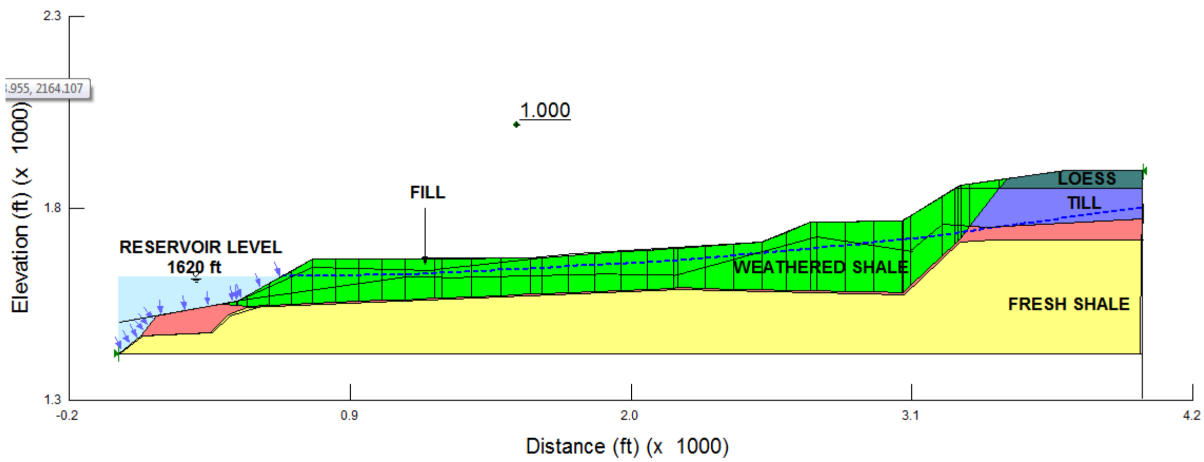

Figure 4-12: Back analyzed section in SLOPE/W showing the obtained factor of safety

Figure 4-11 and Figure 4-12 represent the finite element analysis at the reservoir level of 1620 ft. and the residual friction angle obtained is 7.81°. All the residual friction angle values obtained from the finite element analysis are shown in Table 4-4.

Table 4-4: Summary of Factor of safety values for 2D FEM analysis

Finite Element Method	**Reservoir Level (ft)**	**Residual Friction Angle ($ø_r^{'}$)**	**Factor of Safety (FOS)**
SIGMA/W Stress	1585	6.640°	1
SIGMA/W Stress	1600	7.165°	1
SIGMA/W Stress	1620	7.810°	1

4.4 Three Dimensional Analysis Using Limit Equilibrium Method

In this thesis CLARA/W software is used to conduct the 3D analysis slope stability analysis for the Forest city landslide. CLARA/W is a slope stability analysis software built using limit equilibrium framework and was described in Section 2.5.3. Both 2D and 3D analysis can be performed simultaneously. Morgenstern-Price, Spencer's method, Bishop's simplified method and Janbu methods are available in CLARA/W. Analyses are conducted in all these methods at reservoir elevations of 1585 ft, 1600 ft and 1620 ft only in 3D. The first task in the analysis is to create a 3D profile of the landslide. Figure A-2 represents the plan of the forest city area showing the locations of boreholes for investigation of roadway realignment. To create the 3D profile of the landslide, different cross sections in the range of landslide area are considered. A total of seven cross sections as shown in the Figure A-2 (represented by red colored bold lines) are considered. Among these sections, the section 5 was the same cross section used in 2D analysis in limit equilibrium and finite element methods in sections 4.2 and 4.3 respectively. Hence, profile of the section 5 is directly adopted from the 2D analysis. The profile of the sections 4 and 6 were prepared by using the borehole data from the nearby boreholes. The borehole data for the boreholes specified in this figure are in included in the geotechnical report provided by the Woodward Clyde Consultants (1991). But for sections 1, 2 and 7, no borehole information was available. So, based on the details from the neighboring section i.e. section 3 for 1 and 2, and section 6 for 7 the soil profiles were interpolated.

Figure 4-13 represents the plan of the columns considered in the analysis of the landslide and the red colored solid lines represent the input cross sections same as in Figure A-2. Figure 4-14 to Figure 4-20 show the stratigraphic profiles of the seven sections selected to reasonably represent the extent of the landslide. All the sections were orthogonally interpolated i.e., linear interpolation between each pair of adjacent input points both in Y and X direction (CLARA/W 2001). In all the sections, the blue colored line among the other stratigraphic lines represents the piezometric line at each section. Because of unavailability of ground water levels at all the sections, a uniform ground water table is assumed in all the seven sections. The ground water level shown in Figure 4-14 to Figure 4-20 is related to the reservoir level of 1620 ft. As the input stratigraphic profiles remain constant for other two reservoir levels (1585 ft and 1600 ft) except for the elevation of the piezometric line, they are not documented in this thesis.

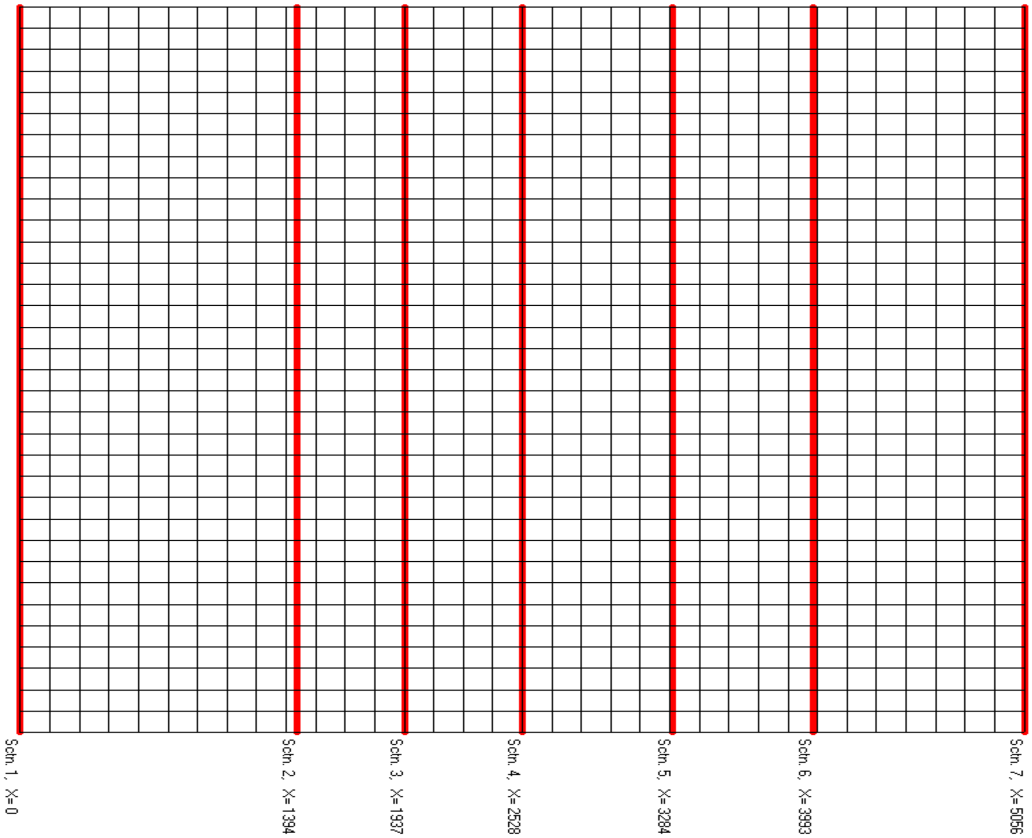

Figure 4-13: Plan of column assembly with input cross sections

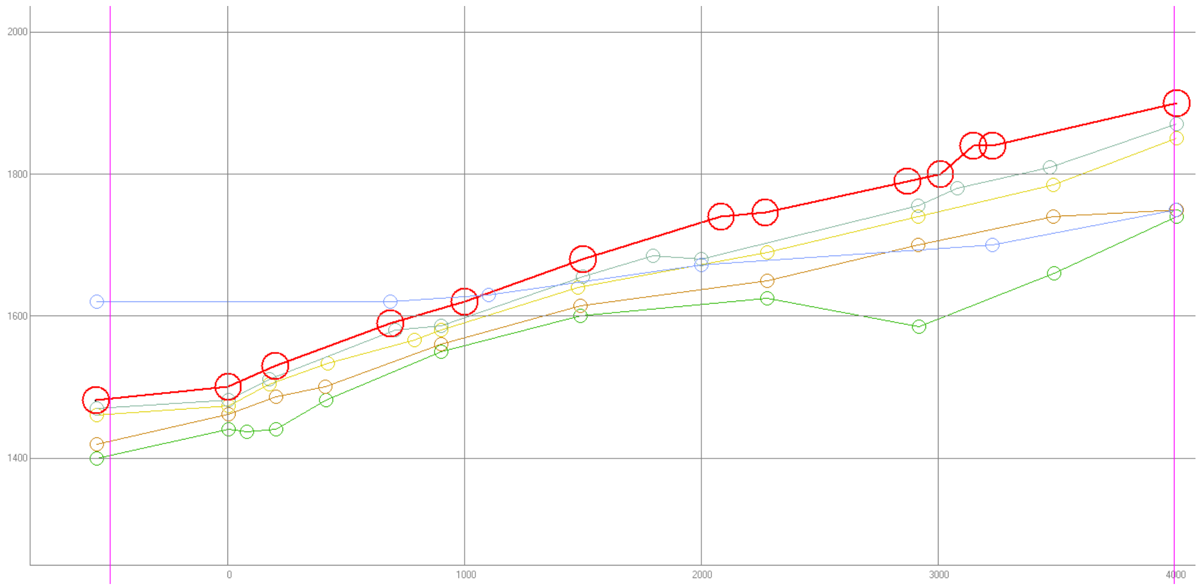

Figure 4-14: Stratigraphic profile of Section 1 in CLARA/W at reservoir elevation of 1585 ft

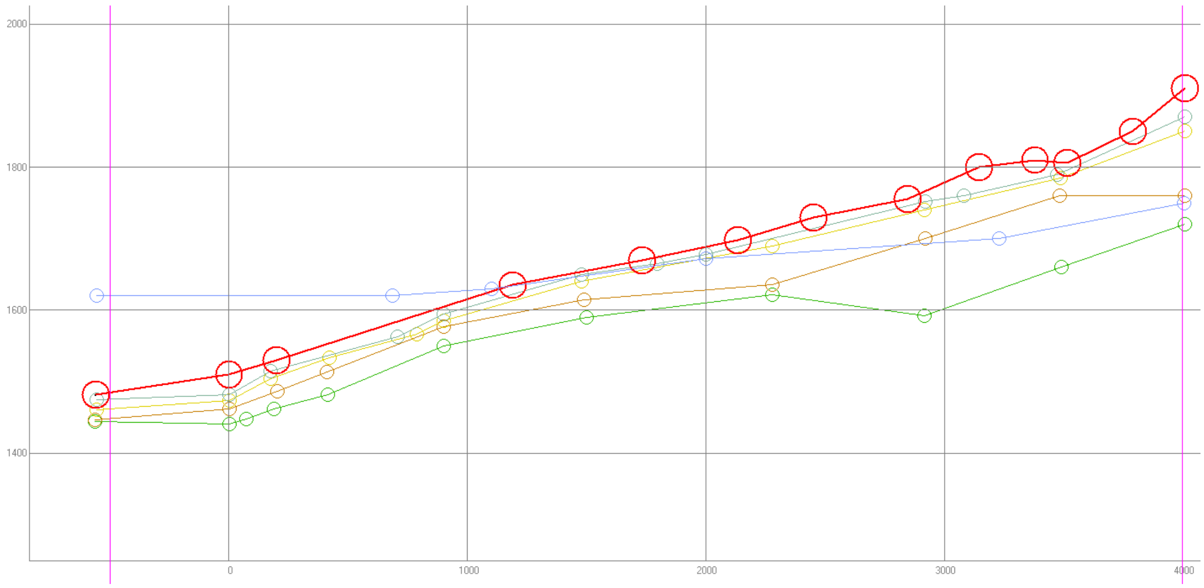

Figure 4-15: Stratigraphic profile of Section 2 in CLARA/W at reservoir elevation of 1620 ft

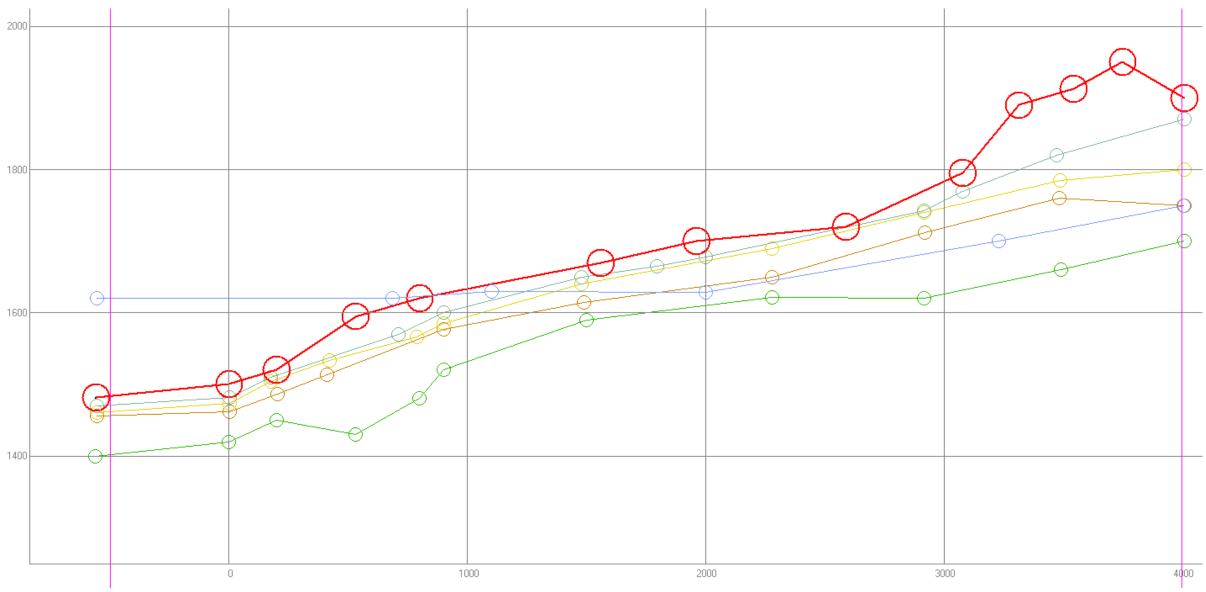

Figure 4-16: Stratigraphic profile of Section 3 in CLARA/W at reservoir elevation of 1620 ft

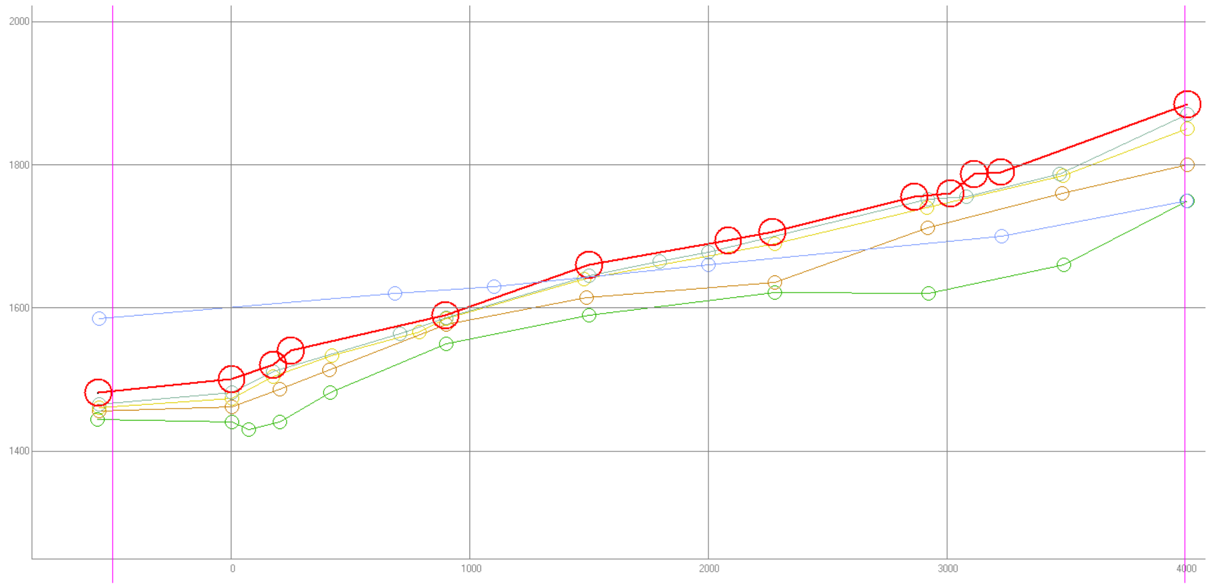

Figure 4-17: Stratigraphic profile of Section 4 in CLARA/W at reservoir elevation of 1620 ft

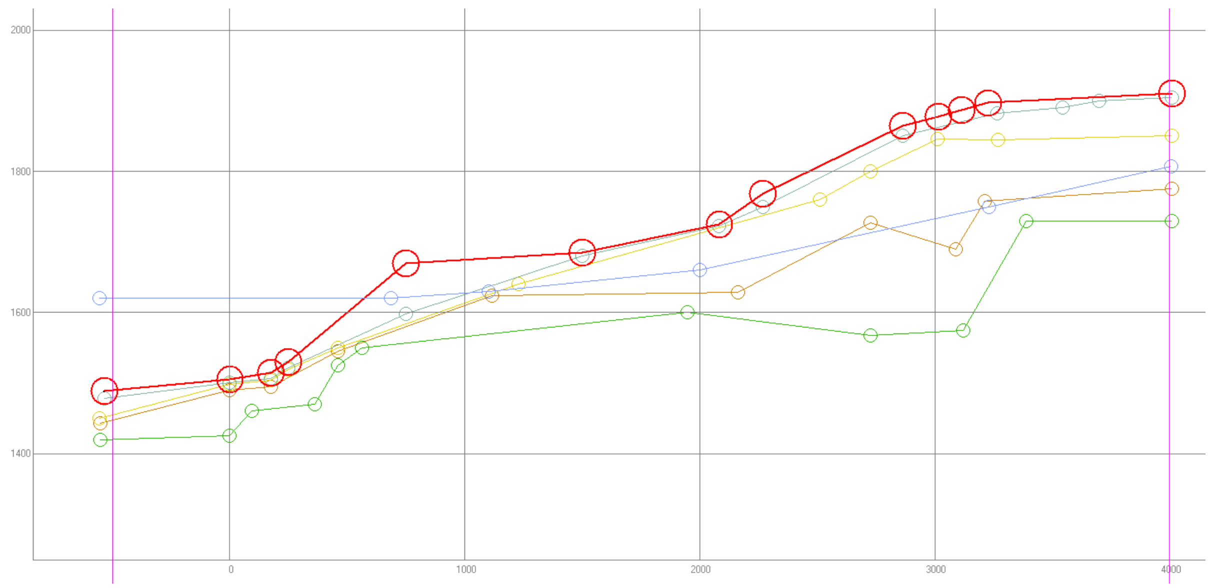

Figure 4-18: Stratigraphic profile of Section 5 in CLARA/W at reservoir elevation of 1620 ft

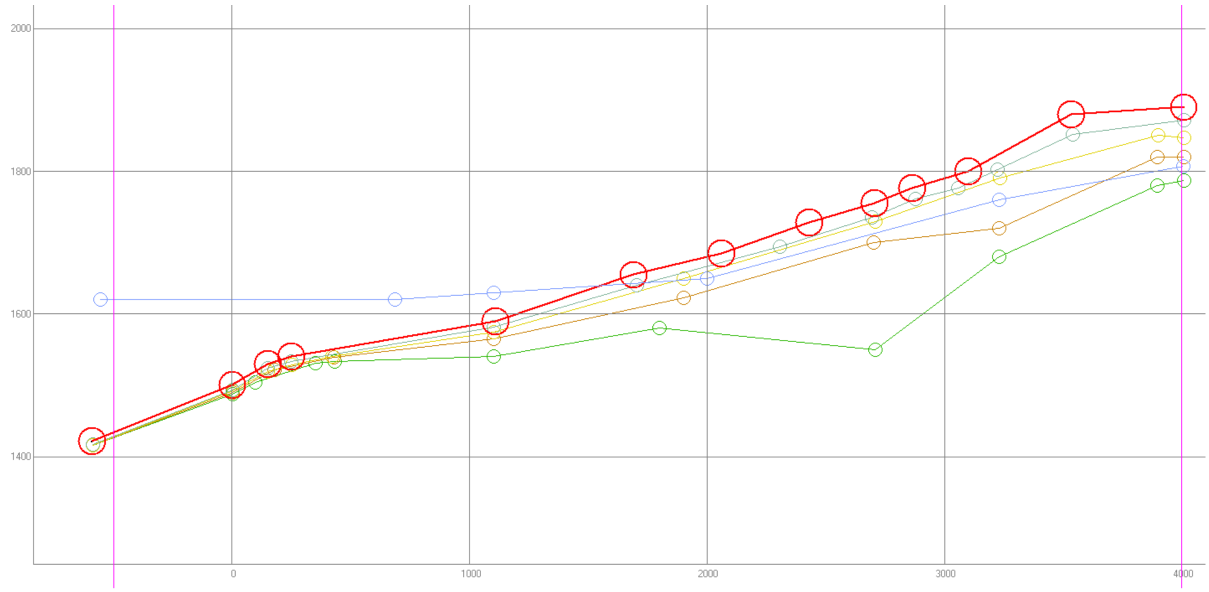

Figure 4-19: Stratigraphic profile of Section 6 in CLARA/W at reservoir elevation of 1620 ft

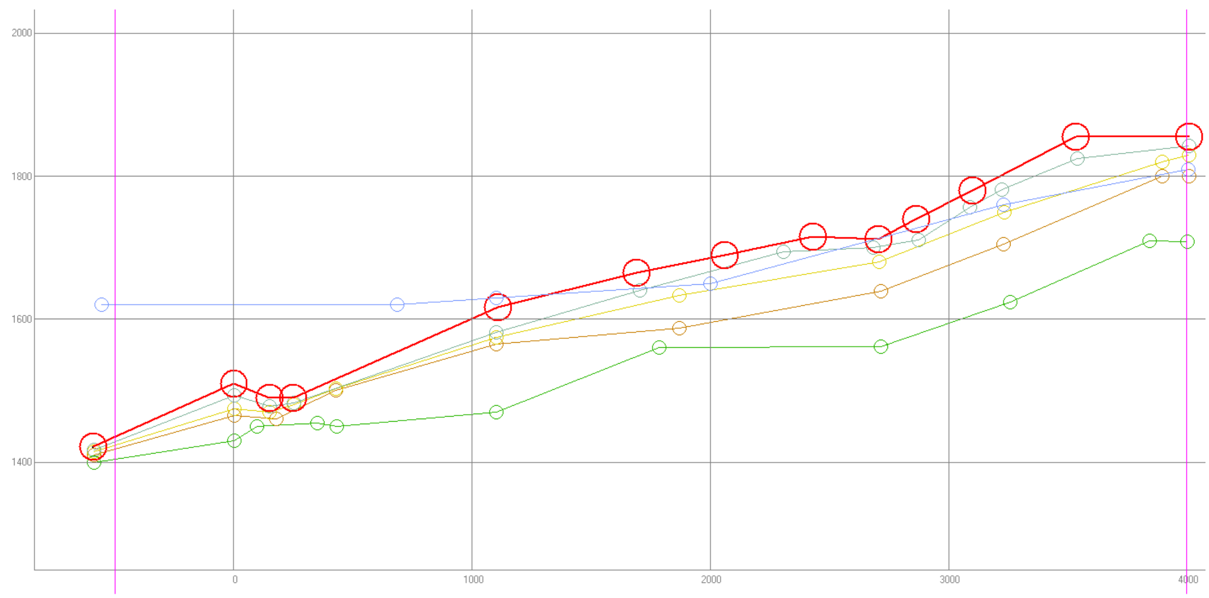

Figure 4-20: Stratigraphic profile of Section7 in CLARA/W at reservoir elevation of 1620 ft

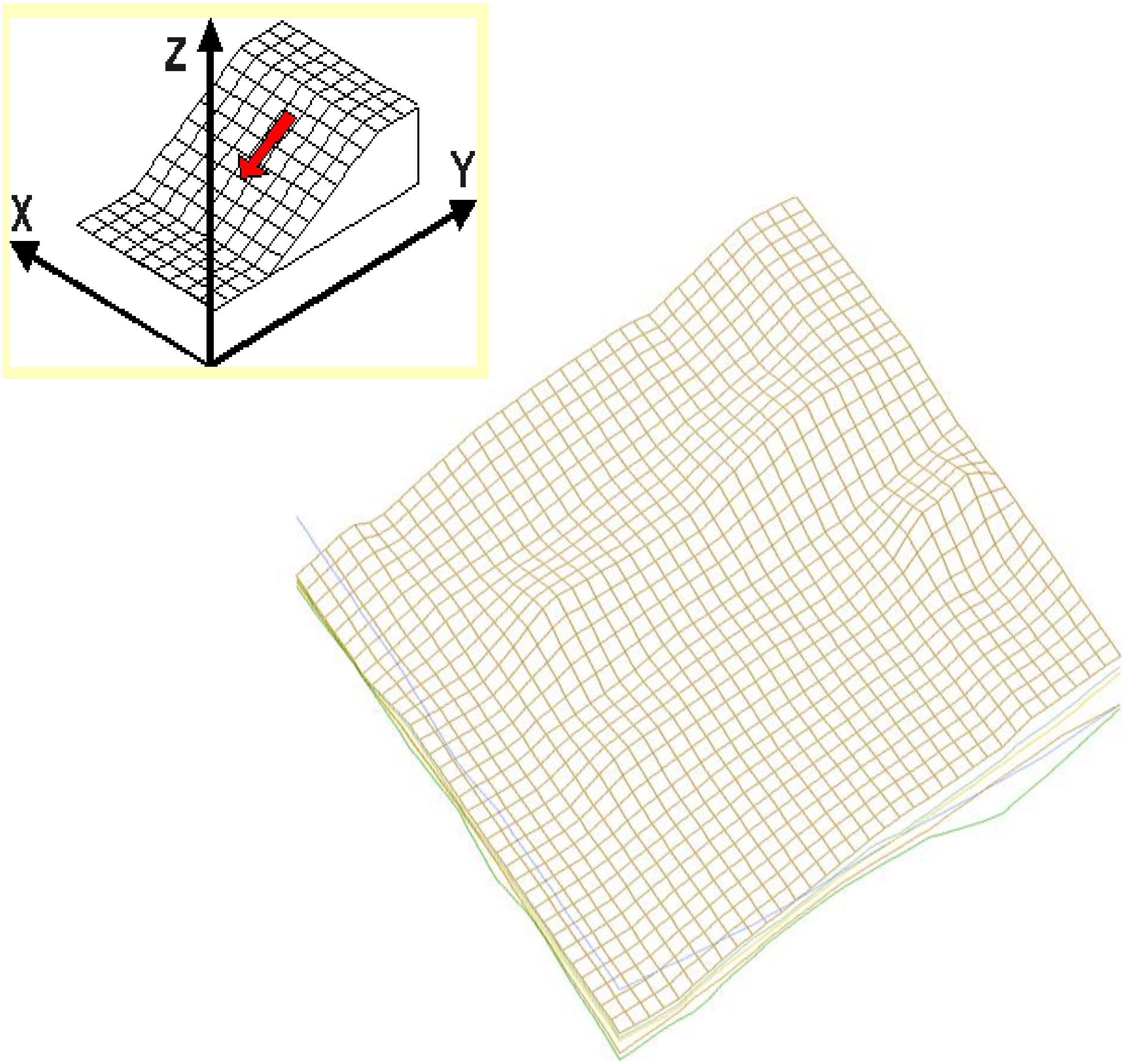

Figure 4-21: 3D slope surface of Forest city landslide created by CLARA/W

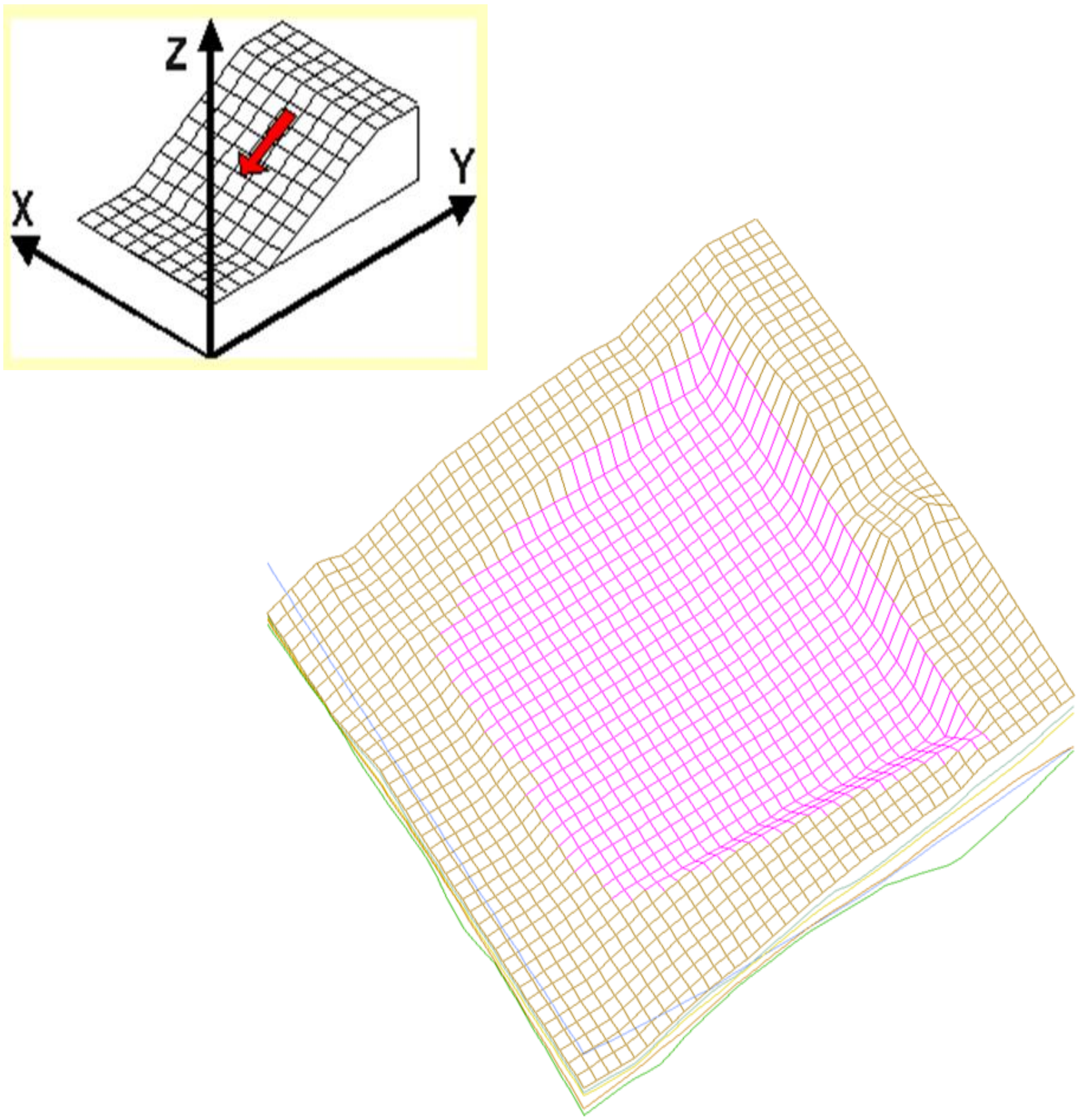

Figure 4-22: 3D model representing the wedged slip surface of Forest city landslide, created by CLARA/W

Figure 4-21 shows the 3D profile of the Forest city slope. In this figure the mesh shown represents the plan of the columns that are considered in the analysis. For 3D analysis the geologic properties for each soil layer are considered to be uniform in all the sections as CLARA/W's framework for geologic parameters of the soil remain constant for all the sections. The geologic properties are the same as considered for the 2D analysis (Table 3-1).The analysis is performed by defining the slip surface at each section. Figure 4-22 represents the failure wedge surface. The area in the pink shaded region represents the entire extent and shape of the Forest city landslide. Figure 4-23 shows the program output of the analysis, using Morgenstern-Price method at a reservoir elevation of 1585 ft. In this, the plan view entire sliding area is shown with contours representing the soil layers. Under the single column output option, the factor of safety for each column in the entire sliding area can be obtained. Also, the factor of safety values obtained for other limit equilibrium methods - Spencer, Bishop and Janbu's method can be obtained simultaneously. Same analysis was repeated by changing the reservoir levels to 1585 ft and 1600 ft. Table **4-5** shows the factor of safety values obtained at each reservoir elevation for different limit equilibrium methods.

Table 4-5: Summary of the Factor of safety values obtained from 3D analysis

Limit Equilibrium Method	Reservoir Level (ft)	Residual Friction Angle ($ø_r'$)	Factor of Safety (FOS)
Bishop's Simplified Method	1585	4.00°	1.00
Bishop's Simplified Method	1600	4.70°	1.00
Bishop's Simplified Method	1620	5.70°	1.00

Bishop's Method
Factor of Safety: 1.00

Slide Volume: 779646100.00
Slide Weight: 100884600000.00
Unbalanced Force: 520605100.0000
Sliding Surface Area: 11251510.00
Number of Active Columns: 1158
Rotation angle: 0.0
Centre Y: 736.06 Z: 13531.69
Negative normal stresses, 0 % weight
Total water thrust
(tension crack): -44320800.00
Total driving moment: 1.318944E+14
Total resisting moment: 1.32461E+14

SINGLE COLUMN OUTPUT
AT X = 2504.7 Y = 2235.6

Angle of Sliding Surface (degrees):
Longitudinal: 5.00
Transverse: 0.00

Total Vertical Stress: 10658.0
Normal Stress: 10641.6
Pore Pressure: 1292.4

Material: Weathered Shale
Friction Angle: 4.0
Cohesion: 0.0

Local F. of S.: 0.80

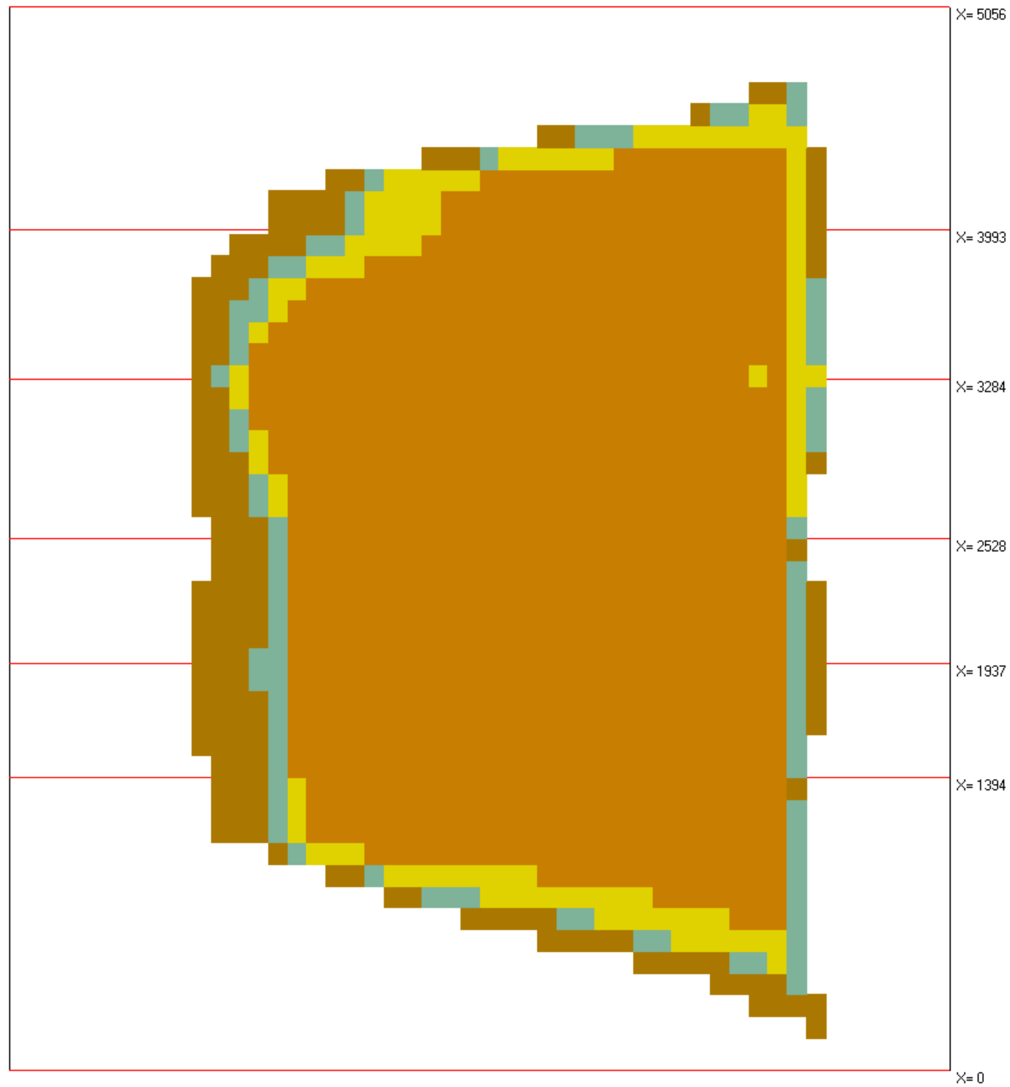

Figure 4-23: Summary of CLARA/W output at reservoir elevation of 1585 ft

4.5 Discussion of the Results

This section consists of discussion on the sensitivity of the residual friction angle in the weathered shale layer to the rise in the reservoir and ground water levels. A comparison of the factor of safety values obtained for the different analytical methods used in this study is included in this part.

4.5.1 Comparison of the residual friction angle obtained in different methods

Residual friction angle for Pierre Shale layer in the Forest City landslide are investigated in different methods of analysis to obtain its realistic ranges of values. Section 4.2 shows the analysis performed by 2D limit equilibrium methods – Morgenstern-Price, Spencer's and Bishop's Simplified methods. Section 4.3 shows the analysis performed by 2D FEM method using ground in situ stresses calculated by SIGMA/W. Section 4.4 shows the analysis results obtained from 3D limit equilibrium method – Bishop's Simplified method. The friction angle shows an 8% increase when reservoir level in increased to 1600 ft from 1585 ft and an increase of 18% when raised to 1620 ft from 1585 ft. The stabilizing effect can be attributed to the trend observed. This trend indicates that the failure might have occurred when the reservoir was at 1585 ft and residual friction angle could be any one of the angle obtained at 1585 ft reported in Table 4-6 depending on the method of analysis. Since the 3D method is known to be more realistic, 4° can be selected as the residual friction angle for the Pierre Shale in this case. Table 4-6 shows the residual friction angles obtained in 2D LEM, 2D FEM and 3D LEM using SLOPE/W, SIGMA/W and CLARA/W respectively. From the results, three important observations are made. First, a consistently increasing trend is observed in the friction angle values as the reservoir level increases. Second, the highest frictional angle value at a particular reservoir elevation is obtained by the 2D Finite element method and the least value is obtained by the 3D limit equilibrium method. Third, comparing 2D analyses methods the friction angle obtained by FEM is more than the friction angle obtained from LEM.

The first trend of increase in friction angle values with a rise in reservoir level can be explained by the fact that the increase in the reservoir level adds counter weight near the toe of the embankment to the moving land mass causing a stabilizing effect. For instance, in case of 2D FEM analysis as shown in Table 4-6 the residual friction angle value increases from 6.64° to 7.165° and to 7.81° as reservoir levels changes to 1585 ft to 1600 ft and to 1620 ft respectively. The friction angle shows an 8% increase when reservoir level in increased to 1600 ft from 1585 ft and an increase of 18% when raised to 1620 ft from 1585 ft. The stabilizing effect can be attributed to the trend observed. This trend indicates that the failure might have occurred when the reservoir was at 1585 ft and residual friction angle could be any one of the angle obtained at 1585 ft reported in Table 4-6 depending on the method of analysis. Since the 3D method is known to be more realistic, 4° can be selected as the residual friction angle for the Pierre Shale in this case.

The second trend is explained as follows - Among the methods of analysis presented in this thesis, the values obtained from the 3D analysis are observed to be low when compared to the 2D analysis results. The reason for this trend is that the 2D analysis does not consider the end effects of the slide and the shear resistance along the sides of the slide mass. Hence, the 2D analysis results are shown to be mostly greater than the 3D analysis results. The 2D analysis results yield a conservative estimate of the strength factors. According to Duncan (1996), Azzouz et al. (1981) and Leshchinsky and Huang (1992) neglecting the 3D effects in the analyses results in a very high back calculated shear strength values. In order discern the extent of the conservatism for this particular case study in terms of the factor of safety obtained from the 3D LEM analysis, a comparison of the factor of safety values obtained for the residual friction angles obtained in 2D LEM and FEM are calculated using CLARA/W software in 3D. The values obtained are tabulated in Table 4-7. Also, it is observed that the relative difference between 2D and 3D factor of safety values is decreasing. This trend is shown in Table 4-7. This could possibly due to the end effects in 3D analysis due to increase in the reservoir levels.

Table 4-6: Summary of the results obtained from the 2D and 3D analysis

Reservoir Level (ft)	Analytical Method	Residual Friction Angle ($ø_r'$)
1585	2D – LEM (Bishop's Method)	5.2°
	2D – FEM (SIGMA/W stress)	6.64°
	3D – LEM (Bishop's Method)	4°
1600	2D – LEM (Bishop's Method)	5.52°
	2D – FEM (SIGMA/W stress)	7.165°
	3D – LEM (Bishop's Method)	4.7°
1620	2D – LEM (Bishop's Method)	6.2°
	2D – FEM (SIGMA/W stress)	7.81°
	3D – LEM (Bishop's Method)	5.7°

Table 4-6 shows that at reservoir level of 1585 ft, the residual friction angle value of 5.2° and 6.64° obtained from 2D LEM and 2D FEM respectively when incorporated in 3D LEM software CLARA/W give a factor of safety of 1.15 and 1.34 respectively. When compared to the original 3D LEM factor of safety value one obtained at $Ø_r' = 4°$, these are 15% and 34% more. So, at this particular reservoir elevation the factor of safety values obtained from 2D LEM and 2D FEM are 15% and 34% more conservative than the factor of safety from 3D LEM analysis. Similarly at the other two reservoir elevations when calculated, the 2D LEM and 2D FEM safety factor values vary by 9% and 29% more from the 3D LEM analysis at 1600 ft reservoir level and by 6% and 23% more at 1620 ft of reservoir level. In general, a 2D analysis is appropriate for a slope stability analysis because it gives a conservative estimate of the factor of safety (Duncan 1996). A 3D analysis is recommended in cases of back analysis for designing of remedial measures for failed slopes (Stark and Eid 1998), slopes with complicated topography, and slopes with complex pore-water pressure condition because effects from the spatial variation of these properties is important in the stability analysis.

In Table 4-8 the factor of safety value for Morgenstern-Price method is back analyzed to be unity and then the residual friction angle values obtained from 2D spencer's method and 2D FEM are incorporated in the same SLOPE/W program and analyzed to get the factor of safety values for comparison purpose. A 3D analysis is not included in this comparison, as convergent solutions cannot be obtained using Morgenstern-Price and Spencer's methods in 3D analysis using CLARA/W. From Table 4-8 it is observed that the variation of factor of safety values between Morgenstern-Price and Spencer's method is 1.6%, 1.1% and 0.6% at 1585 ft, 1600 ft and 1620 ft of reservoir elevations respectively. The variation is minimal and ranges from 0.5% to 1.6%. Hence, this justifies the point that the Morgenstern-Price and Spencer's gives approximately similar results as their assumptions for solving a slope stability analysis problem are the same and satisfy both force equilibrium and moment equilibrium conditions. The factor of safety values between Morgenstern-Price and 2D FEM varies as 6.3 %, 8 % and 5 % for the three reservoir levels. The variation ranges from 5 % to 8 % indicating that the both the methods give approximately similar results. However, the FEM in a slope stability analysis is considered to be accurate (Griffiths and Lane 1999). This variation between the LEM and FEM factor of safety values could be due to different approaches followed by these methods for calculating the stress values.

Table 4-7: Summary of the factor of safety values obtained 3D analysis using CLARA/W

Reservoir level (ft)	**Analytical Method**	**Residual Friction Angle ($ø_r$')**	**FOS (3D LEM – CLARA/W)**
1585	2D LEM (Bishop's Method)	5.2°	1.15
	2D FEM (SIGMA/W stress)	6.64°	1.34
	3D LEM (Bishop's Method)	4°	1.00
1600	2D LEM (Bishop's Method)	5.52°	1.09
	2D FEM (SIGMA/W stress)	7.165°	1.29
	3D LEM (Bishop's Method)	4.7°	1.00
1620	2D LEM (Bishop's Method)	6.2°	1.06
	2D FEM (SIGMA/W stress)	7.81°	1.23
	3D LEM (Bishop's Method)	5.7°	1.00

Laboratory tests were also performed to find out an estimate of the residual friction angle in the weathered shale layer. A comparison of the laboratory values with the values obtained from back analysis reported in this thesis is shown in Table 4-9. The values obtained from the back analysis are approximately 10 % less than the value reported by Schaefer (2002) and varies 2 % to 8 % with the values reported by Bump (1988) This variation between the residual friction angle values could be attributed to the insitu stresses in the field, sample preparation for laboratory testing etc. The value reported by (Grenier and Woodward Consultants1991).is at reservoir elevation of 1585 ft. this value is in good agreement with the values obtained by both 2D LEM and 2D FEM at same reservoir elevation.

Table 4-8: Summary of the factor of safety values obtained from 2D LEM and 2D FEM analyses

Reservoir Level (ft)	Analytical Method	FOS
1585	2D LEM (Morgenstern – Price)	1.000
	2D LEM (Spencer's method)	1.012
	2D FEM (SIGMA/W stress)	1.063
1600	2D LEM (Morgenstern – Price)	1.000
	2D LEM (Spencer's method)	1.011
	2D FEM (SIGMA/W stress)	1.076
1620	2D LEM (Morgenstern – Price)	1.000
	2D LEM (Spencer's method)	1.006
	2D FEM (SIGMA/W stress)	1.049

Table 4-9: Residual friction angles obtained from laboratory testing and back analyses

Reference	Residual friction angle ($ø_r'$)	Test Type
Schaefer & Lohnes (2001)	7.20° to 8.10°	RDS – reversal direct shear test
Bump (1988)	6.40° to 8.00°	RDS – reversal direct shear test
SDDOT (1991)	6.3°	Back analysis - STABL5M
2D LEM – back analysis (in this thesis)	6.21° to 7.40°	2D LEM – back analysis (SLOPE/W)
2D FEM – back analysis (in this thesis)	6.64° to 7.81°	2D FEM – back analysis (SIGMA/W)
3D LEM – back analysis (in this thesis)	4° to 5.7°	3D LEM – back analysis (CLARA/W)

CHAPTER 5. CONCLUSIONS

A parametric study was conducted to investigate the sensitivity of the residual friction angle value to the variation in the reservoir level and the ground water level. The reservoir level and the ground water level were varied proportionately ranging from the normal operating level of the reservoir, 1585 ft. to the highest reservoir level of 1620 ft. Also, the back analysis to obtain the residual friction angle value at different reservoir elevations was performed in 2D LEM (Morgenstern-Price, Spencer's and Simplified Bishop's method), 2D FEM (SIGMA/W stress method) and 3D LEM (Simplified Bishop's method). A comparison of the residual friction angle values obtained from these analytical methods is reported. The following conclusions are based on the 2D and 3D slope stability analyses performed in the parametric study.

1. The increase in the reservoir level has a stabilizing effect on the landslide and the effect is noticed in the increasing trend of the residual friction angle values obtained from the analyses, as the reservoir level increases from 1585 ft. to 1620 ft. This trend indicates that the failure might have caused at the reservoir elevation of 1585 ft. Based on the 2D analysis results the possible friction angle range could be between 6.21° and 6.64°.

2. The variation in the factor of safety values obtained in 3D analysis and 2D analysis is attributed the end effects considered in the 3D analysis approach. A variation of 6 % to 15 % increase in the factor of safety values is observed with 2D LEM –Simplified Bishop's method of analysis and a variation of 30 % increase is observed with 2D FEM of analysis.

3. The factor of safety values obtained using 2D FEM of analysis show an increase of approximately 5 % from the factor of safety values obtained by the 2D LEM.

4. The residual friction angle values obtained from the 2D analysis are in good agreement with the reported laboratory values.

Finally, to select a possible range for residual friction angle value for Pierre shale the values obtained at reservoir level of 1585 ft can be considered because the least strength is obtained at this elevation. So, considering that the failure has triggered at this reservoir elevation the range of friction angles 4° to 6.64° is selected from the 2D and 3D analyses. The friction angle 4° is selected from the 3D analysis and 6.64° is selected from the 2D FEM analysis. But for the design of remedial measures the residual angle obtained from a 3D analysis is known to be more reasonable than the angle from 2D analysis because the 3D analysis considers the 3D end effects. Ignoring these effects makes the friction angle obtained from a 2D analysis to be conservative. The residual friction angle value used by the South Dakota DOT to design remedial measures for this slope was reported to be 6.3° (shown in Table 4-9) at reservoir elevation of 1585 ft. This value is in good agreement with the selected range of 4° to 6.64°.

5.1 Scope for Subsequent Research

The results herein can be extended by,

1. Performing a detailed back analysis using a 3D Finite element method and a comparative study can also be done with the values reported in this thesis to understand the effects of Finite element stresses in 3D.
2. Conducting a study on the variation of pore water pressure as the reservoir level increases by conducting analysis using SEEP/W. Its variation in 2D and 3D can be studied and then its effect on the factor of safety values can be investigated.
3. Performing 3D analysis using other commercial software such as PLAXIS (for FE analysis), FLAC 3D (for Finite difference analysis), ANSYS (for FE analysis) or any commercial FE softwares and 3D analysis can be conducted by other limit equilibrium methods (Only Bishop’s Simplified method is used in this study).
4. A Reliability analysis on the factor of safety and residual friction angle values obtained in this thesis.

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APPENDIX

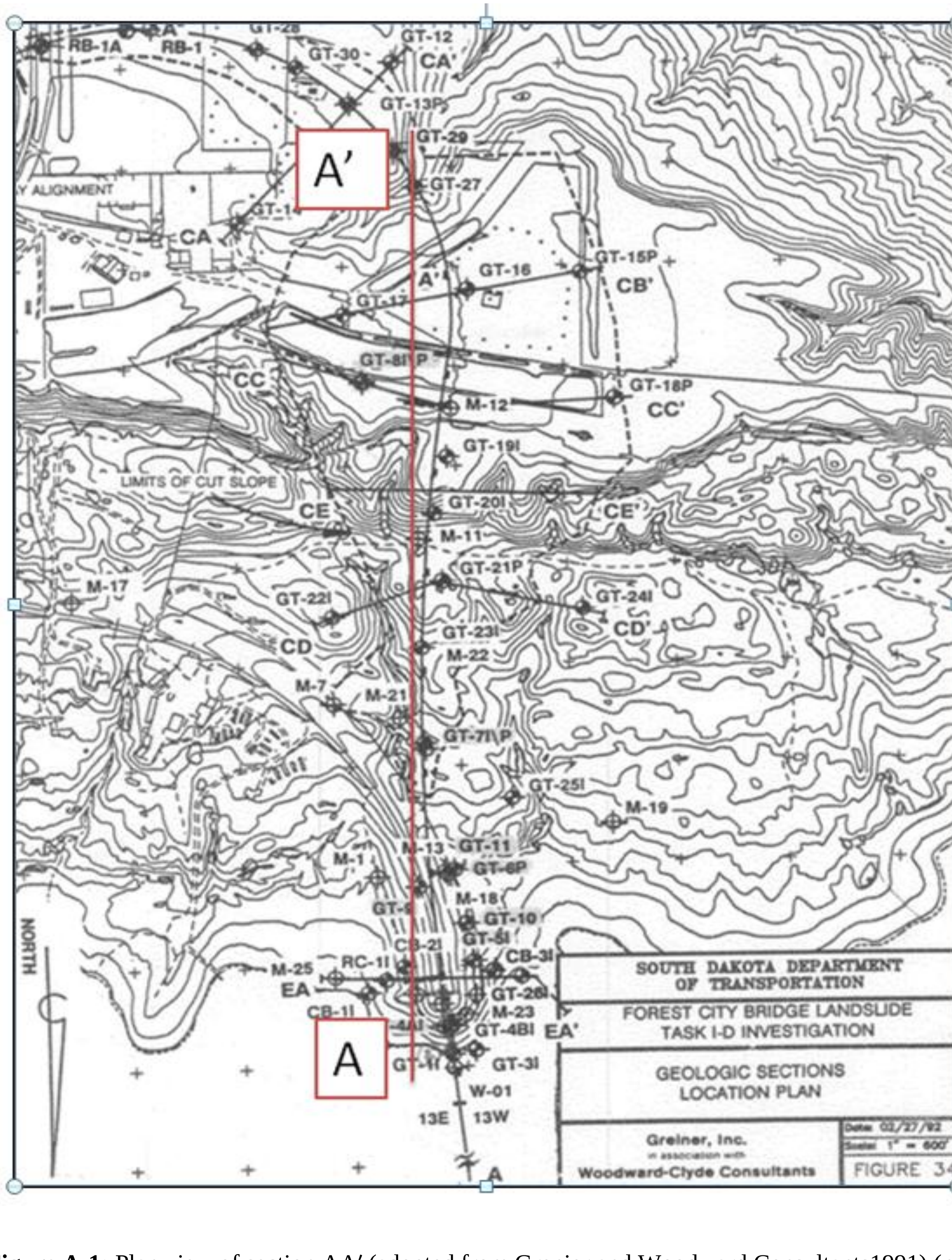

Figure A-1: Plan view of section AA′ (adopted from Grenier and Woodward Consultants1991) (not to scale)

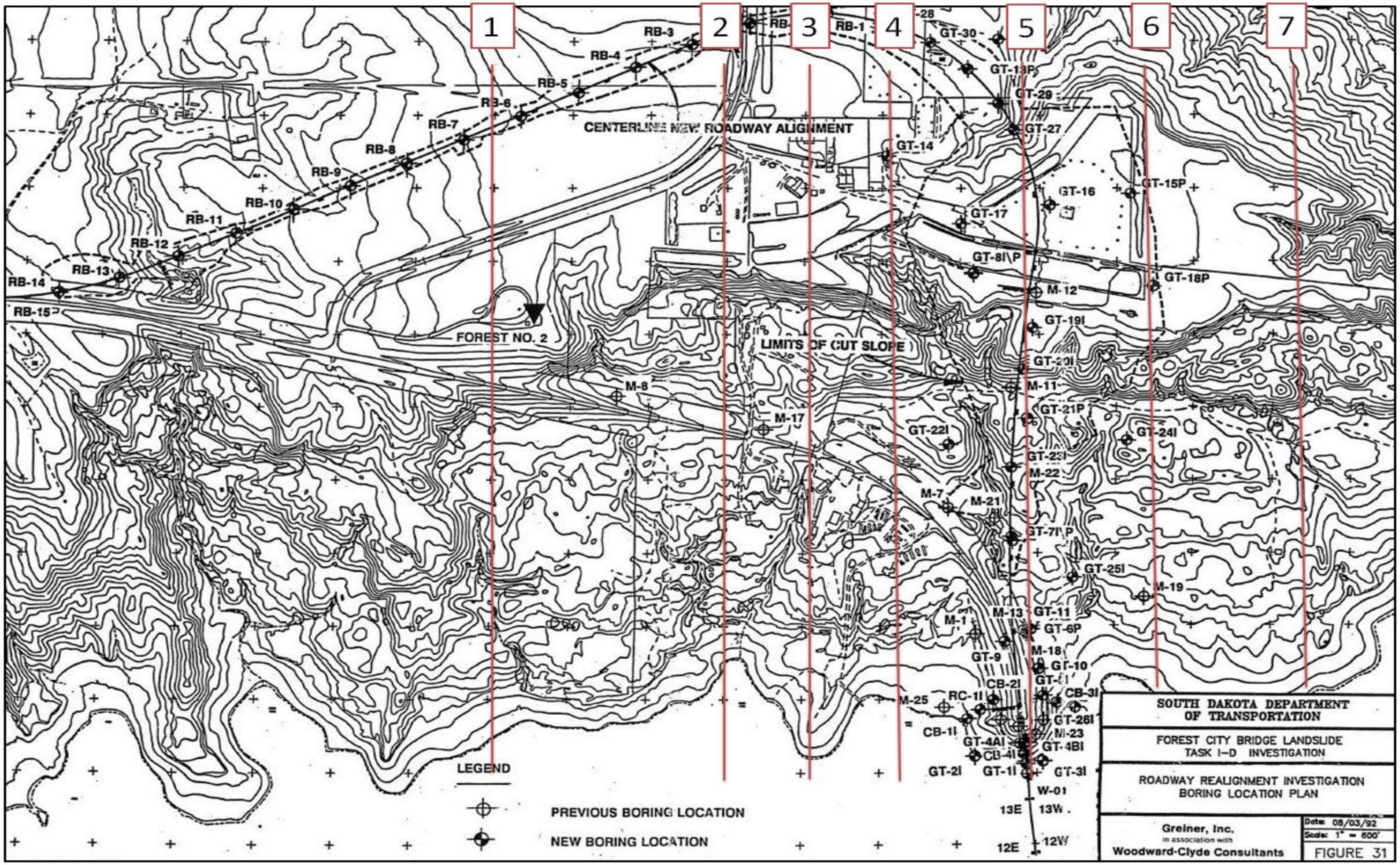

Figure A -2: Plan showing the borehole locations and cross sections selected for the 3D analysis using CLARA/W (adopted from Grenier and Woodward Consultants1991) (not to scale)