

STRUCTURAL GEOLOGY AND ITS INFLUENCE ON THE KINEMATICS OF ROCK EXCAVATION STABILITY: A CRITICAL FOUNDATION CONSIDERATION IN URBAN ENVIRONMENTS

Daniel A. Vellone, PG, M.AEG, M.ASCE, State Geologist, USDA-Natural Resources Conservation Service, Geological Services Unit – Northeast Region, Holden, MA; daniel.vellone@ma.usda.gov

Charles Merguerian, Ph.D., Principal, Duke Geological Laboratory, 55 Spongia Road, Stone Ridge, NY 12484; charlesm@dukelabs.com, www.dukelabs.com and Professor of Geology, Hofstra University, Hempstead, NY 11549; charles.merguerian@hofstra.edu

Abstract: Urban construction environments in rock present unique foundation challenges. Owners, design professionals, and contractors must be increasingly aware of potential site constraints and limitations when extending foundation excavations to a project's physical limits. Excavations designed to extend the limits of the site footprint to maximize prime real estate settings and building floor-space often require excavation support or underpinning for adjacent structures, foundation elements and infrastructure including utilities, retaining walls supporting sidewalks, roadways and other areas in common rights-of-way.

The structural geology underlying a project's footprint has a direct and controlling influence on efficient and low-risk rock excavation. Structural geologic features such as the foliation parting and crosscutting discontinuities (shears, faults, and joints) in metamorphic rocks control the short- and long-term stability of rock excavations. Stability should be evaluated through comprehensive kinematic analysis in the design phase, well before the first piece of excavation equipment is mobilized to the site. Routinely, the consideration of a rock excavation's structural geology is left until problems arise during construction, manifesting in excessive over-break, rock slides and wedge-type fallout requiring remedial support and underpinning or potential differing site condition claims.

Not a typical presentation of a case study, the participant will gain an understanding of the types of structural geologic features which often govern the stability of foundation excavations in urban environments through the explanation of engineering kinematic analysis of structural geology data. Select project case histories presenting structural geological data from projects constructed in urban environments in New York and Massachusetts will be presented to emphasize learning objectives and to illustrate wedge-stability analysis as applied to excavations in foliated metamorphic rock.

I INTRODUCTION

Construction in urban environments present unique and often complex foundation challenges when excavations are to be made in rock. The ultimate stability of excavations in rock is governed by the structural geology of the excavated rock mass. The geologic character of the rock is significantly influenced by naturally occurring geologic features such as foliation parting and other crosscutting features (shears, faults, and joints) which may be collectively termed "discontinuities." Discontinuities are generally planes or zones of weakness through a stronger intact rock mass, so failure tends to occur preferentially along these surfaces. As such, discontinuities directly govern the appearance and stability of the final rock cut.

The overall purpose of performing a geological assessment and mapping program is to define the set (or sets) of discontinuities which may control stability of a particular excavation. While the orientation of discontinuities are commonly the prime geological factor influencing stability, other properties such as the persistence, spacing, roughness and infilling are significant considerations for the evaluation of the final

excavation stability. The goal of the characterization process is to evaluate the overall physical characteristics of the rock mass that have the greatest influence on excavation behavior.

II GENERALIZED BEDROCK GEOLOGY OF THE NEW YORK CITY METROPOLITAN REGION

New York City is situated at the extreme southern end of the Manhattan Prong (Figure 1), a northeast-trending, extensively folded, deeply eroded sequence of metamorphosed Proterozoic to Lower Paleozoic rocks that widen northeastward into the crystalline terrains of New England (Schuberth, 1968; Baskerville, 1989; Isachsen et al., 2000; Merguerian, 2005a,b). The New York City Metropolitan area lies at the convergence of three physiographic provinces - the New England Upland to the northeast, the Newark Triassic-Jurassic Basin (Triassic Lowland) on the west and southwest, and the Atlantic Coastal Plain to the south (Baskerville, 1989). Southward from New York City, the rocks of the Manhattan Prong plunge unconformably beneath predominately buried Mesozoic rocks, Cretaceous strata, and overlying Pleistocene (glacial) sediment that cap Long Island and much of Staten Island.



Figure 1 – Physiographic diagram of the tri-state region showing the major physiographic provinces. (From Bennington, J. B., and Merguerian, Charles, 2007, *Geology of New York and New Jersey*: Thomson Brooks/Cole, 24 p.)

The Manhattan Prong underlies a landscape of rolling hills and valleys whose configurations are closely controlled by the structure and lithology of the underlying bedrock. Figure 2 shows a generalized geologic map of New York City bedrock, consisting of metamorphic, meta-igneous, and igneous rock ranging from Proterozoic to Ordovician age. The hilly terrains are underlain by strata that are generally resistant to erosion (Fordham Gneiss, Yonkers Gneiss and by various schists and gneisses of the Manhattan, Walloomsac, and Hartland formations). Because of differential weathering susceptibility the

valleys are generally underlain by brittle faults, fault zones or the presence of metacarbonate rocks of the Inwood Marble formation.

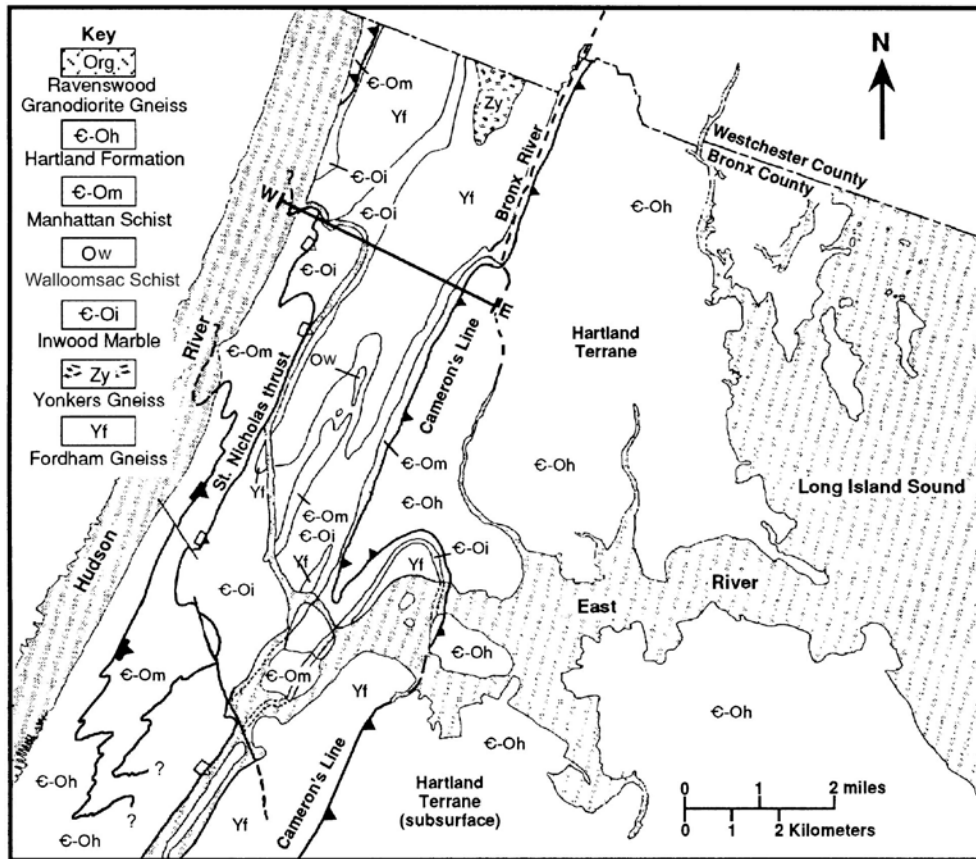


Figure 2 – Geological map of New York City showing the generalized structural geology of the region. Triangles show the dip of Cameron’s Line (solid) and the St. Nicholas thrust (open) and the flagged triangles indicate overturned thrusts. Most faults and intrusive rocks have been omitted. Adapted from Merguerian and Baskerville (1987) and Merguerian and Merguerian (2004).

Rocks of the New York City Metropolitan region are characterized by diverse age, mineralogy, and metamorphic grade and exhibit complex geological structures. As a result, the engineering properties of the different formations are variable. The structures are dominated by crosscutting surfaces of foliation and gneissic layering formed by the preferential alignment of platy and linear minerals within the rock. All of the features that are mapped in the field (composition, texture, metamorphic grade, fabric orientation, and mica content) have also been observed to control means and methods in shallow construction excavations (Merguerian, 2008; Vellone and Isler, 2008).

In New York City, the basement crystalline rock consists of the Proterozoic Z Fordham Gneiss. This substrate was overlain by the carbonate protoliths of the Cambro-Ordovician Inwood Marble, and above it the Ordovician Walloomsac Schist. During the Taconic Orogeny the Cambro-Ordovician Manhattan Schist was thrust over this sequence, and the Hartland Formation was overthrust above that along Cameron’s Line (Merguerian, 1996). Slivers of serpentinite were wedged into and between the Hartland and Manhattan formations (Merguerian and Moss, 2005) and all of these units were regionally metamorphosed and intruded at various times by granites (Baskerville, 1989; Merguerian, 2008).

About 450 million years ago, during the Taconic orogeny, the rocks of the Manhattan Prong were tightly folded and metamorphosed (Isachsen et al., 2000). Consequently, complex fold and fracture patterns have resulted from many stages of polydeformation. The geologic structure of metamorphic bedrock is typically dominated by surfaces of foliation and gneissic layering formed by the preferential alignment of platy minerals within the rock.

The bedrock of New York City was initially metamorphosed under granulite- to amphibolite facies conditions. Proterozoic mafic to intermediate metaplutonic rocks harbor garnet+pyroxene bearing assemblages and create dense, hard rocks known as granulites (Brock, Brock, and Merguerian, 2001; Merguerian, 2008). In formally clay-rich Paleozoic strata metamorphism resulted in the growth of oriented mica along with kyanite and sillimanite in a matrix of flattened and recrystallized quartz, plagioclase, and garnet to produce a foliated schistose to gneissic rock mass. Interlayered former sandy units recrystallized into mica-poor layers (granofels) consisting of intergrown quartz and plagioclase with minor mica. Intercalated mafic volcanic rocks recrystallized into foliated amphibole and plagioclase rich rocks (amphibolite).

A. Fordham Gneiss (Yf)

The Fordham Gneiss constitutes the oldest underpinning of rock formations in the New York City area and consists of a complex assemblage of Proterozoic Z metaplutonic, metasedimentary and granitoid rocks. The Fordham consists of predominately massive gneiss with subordinate schistose rocks. These metaigneous rocks developed coarse-textured fabrics during Grenvillian granulite facies metamorphism, and retained their nearly anhydrous, poorly foliated character during subsequent high-grade Ordovician and younger deformation and retrograde metamorphism (Merguerian, 2005a,b; Merguerian, 2008). They have been metamorphosed to a rock consisting of quartz, feldspar, and mica with varying amounts of amphibole, pyroxene, plagioclase, and garnet that has locally resisted hornblende and biotite grade retrograde regional metamorphism. The formation is generally very hard and resistant to weathering.

B. Inwood Marble (€-Oi)

The Inwood consists of typically white to bluish-gray to brownish coarse- to fine-textured calcitic and dolomitic marble locally with siliceous interlayers containing tremolite, phlogopite, actinolite, quartz, diopside, graphite, and dravite (Merguerian, Merguerian, and Cherukupalli, 2011). The pure carbonate and siliceous layers are frequently massive but the micaceous zones tend to be more foliated and schistose. The formation is prone to weathering. In many places, the Walloomsac Formation can be found interlayered with the underlying Inwood indicating a former depositional sedimentary contact.

C. Walloomsac Schist (Ow)

This discontinuous unit is composed of fissile brown- to rusty-weathering, fine- to medium-textured quartz-feldspar-biotite-muscovite-kyanite-sillimanite-garnet-pyrite-graphite schist and migmatitic schist containing interlayers centimeters to meters thick of plagioclase-quartz-muscovite granofels and layers of ("Balmville") calcite marble and calc-silicate rock (Merguerian, 2008). Garnet occurs as porphyroblasts up to 1 centimeter in size and amphibolite is absent. Strongly pleochroic reddish titaniferous biotite, garnet, graphite, and pyrite are diagnostic petrographic features of the formation. While the schistose layers are well foliated, the marble and calc-silicates tend to be more massive.

The Walloomsac Schist is interpreted as being autochthonous (depositionally above the Inwood Marble and underlying Fordham Gneiss), and is assigned a middle Ordovician age (Merguerian, 2005a,b; Merguerian, 2008). The lack of amphibolite and the presence of graphitic schist and quartz-feldspar granofels enables the interpretation that the Walloomsac Schist is the metamorphosed equivalent of carbonaceous shale and interlayered greywacke and is therefore correlative with parts

of the middle Ordovician Annsville and Normanskill formations of SE New York and the Martinsburg formation of eastern Pennsylvania (Merguerian, 2005a,b; Merguerian, 2008).

D. Manhattan Schist (Є–Om)

The Manhattan Schist is of Late Proterozoic to Ordovician age in New England. These rocks, which contain calc-silicate interlayers in western Connecticut, are inferred to represent metamorphosed sedimentary- and minor volcanic rocks deposited in the transitional slope- and rise environment of the Early Paleozoic continental margin of ancestral North America (Merguerian, 1983, 2008). The Manhattan consists of very massive rusty- to sometimes maroon- weathering, medium- to coarse-textured, biotite-muscovite-plagioclase-quartz-garnet-kyanite-sillimanite gneiss and, to a lesser degree, schist. The unit is characterized by the lack of internal layering, however it does have present layers and lenses up to 10 centimeters thick of kyanite+sillimanite+quartz+magnetite, centimeter to meter scale layers of blackish amphibolite, and scarce quartzose granofels. The unit is a major ridge former in northern Manhattan, a testament to its durability to weathering owing to the lack of layering and presence of resistant minerals garnet, kyanite, and sillimanite.

E. Hartland Schist (Є – Oh)

The Hartland represents metamorphosed deep-oceanic shale, interstratified graywacke, and volcanic rocks formed offshore adjacent to North America during Late Proterozoic to Early Paleozoic time (Baskerville, 1989; Isachsen et al., 2000). The Hartland formation is dominantly gray-weathering, fine- to coarse-textured, well-layered and foliated muscovite-quartz-biotite-plagioclase-kyanite-garnet schist, and gneiss with centimeter and meter scale layers of gray quartzose granofels and greenish amphibolite±garnet (Merguerian, 2008). Compositional layering is preserved in the Hartland forming a well-layered metamorphic rock mass consisting of intercalated schist, granofels, and amphibolite. The Hartland Schist is relatively easy to excavate because of its pervasive foliation and presence of internal compositional layering. The high mica content, tends to make for a comparatively softer rock.

F. Serpentinite (Proterozoic(?) to Paleozoic)

The serpentinites are black to greenish fine-grained rocks containing serpentine group minerals including chrysotile, chromite, magnetite, orthoamphibole, magnesite, talc, calcite, chlorite, and relict olivine and pyroxene. The rock ranges from massive to schistose (Merguerian and Moss, 2005).

G. Pegmatites (Proterozoic to Paleozoic)

All units of the New York City bedrock described above are intruded by granitoids that range from sheared pre- and syn-tectonic intrusives to post-tectonic bodies (Baskerville, 1989; Merguerian, 2005a,b; Merguerian, 2008). They range from fine-textured to pegmatitic and occur as dikes, sills, stocks, and small plutons consisting of essential K-feldspar (microcline and orthoclase) and quartz together with varying amounts of plagioclase, biotite, hornblende, muscovite, and subordinate garnet.

III ANALYSIS OF ROCK MASS CHARACTERISTICS

An analysis of the rock mass characteristics must be made in order to accurately evaluate the stability of an excavation in rock. The rock quality designation (RQD of Deere et al., 1967) is a basic component of many rock mass classification systems for engineering purposes. The RQD is the quantitative measure of the cumulative length of rock core fragments exceeding 4 inches in length divided by the total length of the rock core during each core segment. A low RQD value would be indicative of a closely fractured rock, while an RQD of 100 percent means that all pieces of core are longer than 4 inches. The Rock Mass Rating (RMR) system provides a means for estimating the strength of rock masses based upon the

parameters of uniaxial compressive strength (UCS), RQD, spacing, condition and orientation of discontinuities and groundwater considerations. The condition of the rock mass may be qualitatively classified on the basis of these parameters from 'very poor' to 'very good' rock.

A Brunton geological compass may be used to measure dip and dip direction of prominent structural discontinuities. Field observations should also be made of rock composition, degree of weathering, discontinuity spacing and lengths, how the discontinuities are terminated, and roughness and infilling characteristics. The degree and intensity of the field investigation is directly related to the complexity of the project. Often preliminary mapping is completed with subsequent field investigation to further define discontinuity sets and fill data gaps. As the quality of the analysis is only as good as the field data it is based upon, an experienced geologist should be tasked with overseeing the field investigation program.

The Hoek-Brown failure criterion may be used to determine rock mass strength characteristics (Hoek, Carranza-Torres and Corkum, 2002). A critical component in determining rock mass strength using the Hoek-Brown failure criterion is the UCS of intact pieces of rock that make up the rock mass. However, the UCS of rock may be strongly influenced by the density, distribution and connectivity of weak micro-structural elements (Aydin, 2009). While the performance of laboratory testing on rock core samples to determine UCS is recommended, field estimates of UCS may be made from qualitative observation using a Schmidt rebound hammer (Vellone, 2009; see Hudson and Harrison, 1997, pg. 180). A material constant, m_i , related to the frictional characteristics of the rock material is required for the evaluation of the rock mass strength using the Hoek-Brown criterion. While this parameter may be determined through laboratory triaxial testing, these tests are infrequently carried out for small-scale projects. As a result, standard practice is to estimate the value from a published table of typical values (Hoek and Guevara, 2009).

A Schmidt rebound hammer may be used on New York City rock to determine an approximate measurement of surface hardness for correlation to other rock mass properties (Vellone and Merguerian, 2007). Despite its apparent simplicity, the Schmidt hammer has proven to be one of the most useful indicators of rock strength (Hudson and Harrison, 2000). The Schmidt hammer consists of a spring-loaded steel piston-type hammer, which works on the principle that the rebound of an elastic mass impacting on the tested surface is a function of the hardness of the surface itself (Aydin, 2009). The accuracy of the hammer should be verified on a standard calibration anvil in accordance with manufacturer's recommendations prior to field use and rebound values should be recorded as recommended by ASTM D5873. Data correction methods may be used to normalize data (Basu and Aydin, 2004; Shorey et al., 1984) and descriptive statistics including *mean*, *median* and *mode* values should be presented to fully express variations in surface hardness.

A Type-N hammer is preferred in field applications and has been shown to be less sensitive to surface irregularities (Vellone, 2009). The higher impact energy of the Type-N hammer corresponds to probing a larger volume of material from a deeper and wider penetration, thereby reducing scatter in rebound values obtained at different points on a heterogeneous surface (Aydin and Basu, 2005). Schmidt hammer tests should be performed in the horizontal direction ($\theta=0$) and normal to any apparent schistosity within the rock fabric. Planes of anisotropy in foliated metamorphic rocks control the response to impact and loading. Rebound values may be strongly reduced when the impact direction is normal to foliation planes as they absorb impact energy whereas the UCS values steeply decrease at oblique angles of anisotropy. Low and high rebound readings are considered equally necessary to reflect the nature of heterogeneity and potential spread in the values of mechanical properties.

IV KINEMATIC ANALYSIS OF ROCK MASS STABILITY

In order to optimize the excavation and support methods it is necessary to consider how the rock mass will first respond to the excavation without considering support. These structural geological features that influence rock cut stability often occur in three dimensions with a degree of natural scatter. In order to use

the data to evaluate the stability of the cut, it is necessary to analyze the data using stereographic projection to identify discontinuity sets, and examine their influence on excavation stability. The stereographic projection method allows the three-dimensional orientation data to be represented and analyzed in two dimensions. As noted, all natural discontinuities have a certain amount of variation in their orientations that result in scatter of the pole plots using stereographic analysis. However, by contouring the plot, the most highly concentrated areas of poles can be more readily identified, thereby identifying an *arithmetic mean* orientation of the discontinuity dip and dip direction for use in further analyses of excavation stability. These analyses consider kinematically possible, structurally controlled failures within a rock mass, as opposed to non-structurally controlled failures in which some or all of the failure surface passes through intact rock.

A. Toppling Stability Analysis

A toppling analysis may be performed using stereographic projection based upon the variability cones indicating the extent of the joint set population, a slip limit based upon the joint friction angle and cut slope angle and kinematic considerations. As previously discussed, an equivalent Mohr-Coulomb friction angle may be calculated using the Hoek-Brown Criterion based upon the intact UCS of the rock determined through laboratory testing or field estimates using the Schmidt rebound hammer. The friction angle is represented by a friction cone with limits set relative to the rock excavation face dip direction, as suggested by Goodman (1989). The zone bounded by these curves delineates the toppling failure region. Poles plotted within this region represent a toppling risk (see Figure 3).

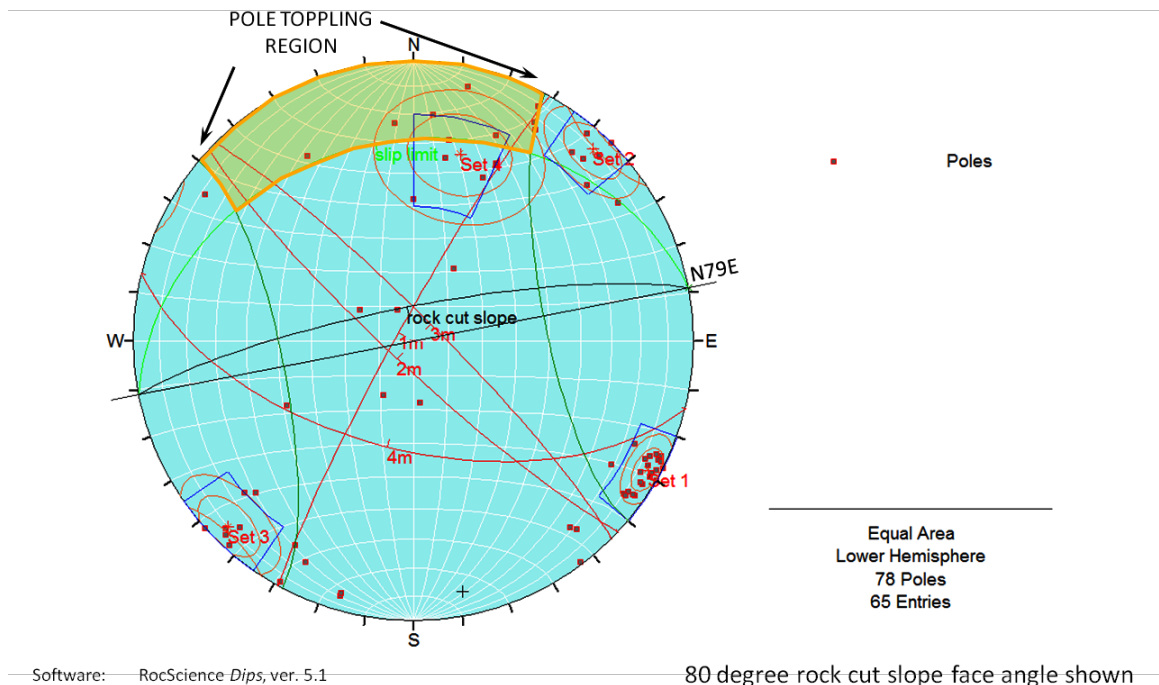


Figure 3 – Kinematic Analysis of Toppling Stability

While the rock excavation alignment is represented by a cut face of N79°E (dip direction 349 degrees) in this example, relatively few poles, generally corresponding to Set 4, plot within the pole toppling region indicating a relatively low potential risk of instability through the toppling failure mode. However, it may be realized that the potential risk of instability would increase significantly under an

alternate scenario if the rock excavation alignment cut face was oriented at N110°E (dip direction 20 degrees) engaging the poles corresponding to both Set 4 and Set 2 discontinuities.

B. Planar Sliding Stability Analysis

A planar sliding analysis may be performed using stereographic projection based upon the variability cones indicating the extent of the joint set population, a friction cone, and a daylight envelope to test for the combined frictional and kinematic possibility of planar sliding. A daylight envelope is added to the stereographic projection to test for kinematics (i.e., a rock slab must have somewhere to slide into – free space created by the rock excavation). Any pole which falls within this envelope is kinematically free to slide if frictionally unstable. A pole friction angle was added to the stereographic projection based upon the equivalent Mohr-Coulomb friction angle. As previously noted, this friction angle may be calculated using the Hoek-Brown Criterion based upon the intact UCS of the rock determined through laboratory testing and validated in the field using the Schmidt rebound hammer. Any pole represented on the stereographic projection falling outside of this cone represents a plane which could slide if kinematically possible. The crescent shaped zone formed by the intersection of the daylight envelope and the pole friction circle encloses the region of planar sliding. Any poles within this region represent planes which can potentially slide (see Figure 4).

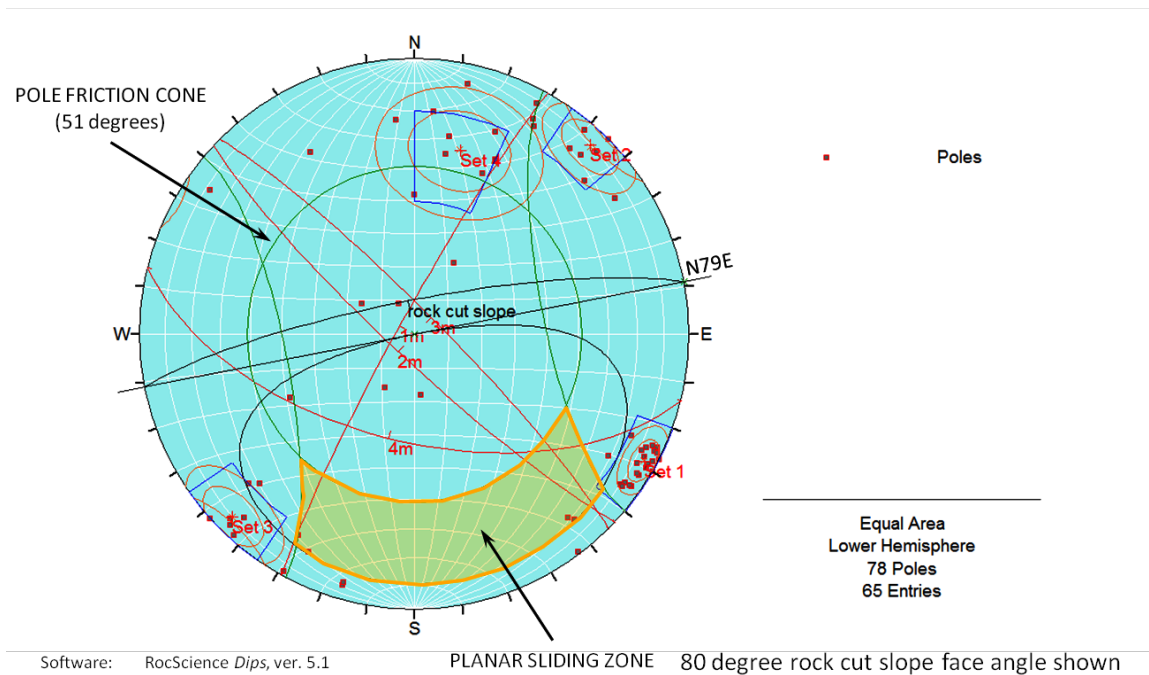
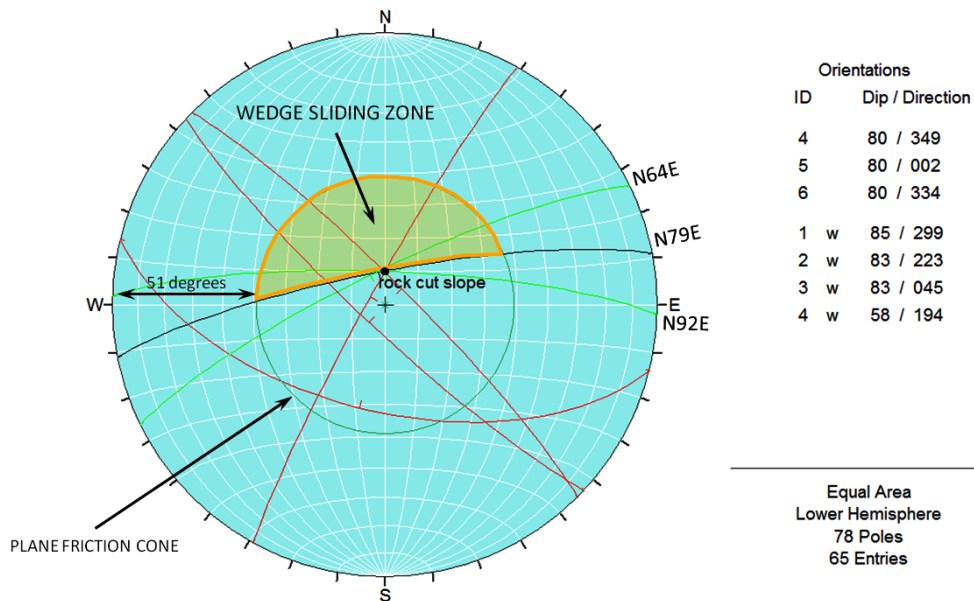


Figure 4 – Kinematic Analysis of Planar Sliding Stability

While the rock excavation alignment is represented by a cut face of N79°E (dip direction 349 degrees) in this example, relatively few poles, corresponding to Set 1 and Set 3, plot within the pole toppling region indicating a relatively low potential risk of instability through the toppling failure mode with the given rock cut alignment. However, it may be realized that the potential risk of instability would increase significantly under an alternate scenario if the rock excavation alignment cut face was oriented at N30°E (dip direction 300 degrees) engaging the poles corresponding to Set 1 discontinuities. Similarly, the potential risk of instability would increase significantly under an alternate scenario if the rock excavation alignment cut face was oriented at N135°E (dip direction 45 degrees) engaging the poles corresponding to Set 3 discontinuities.

C. Wedge Sliding Stability Analysis

Multiple joints can form tetrahedral blocks (wedge geometries) that can slide along lines of intersection. A wedge sliding analysis may be performed using the stereographic projection using a plot of the major planes and a plane friction cone. A plane friction angle was added to the stereographic projection based upon the equivalent Mohr-Coulomb friction angle. As previously noted, this friction angle may be calculated using the Hoek-Brown Criterion based upon the intact UCS of the rock determined through laboratory testing and validated in the field using the Schmidt rebound hammer. The wedge sliding zone is represented by the crescent shaped region formed by the intersection of the plane friction cone and the cut slope face (see Figure 5A). The formation of tetrahedral blocks and wedge sliding may occur if the mean joint set orientation intersections fall within an area defined by the friction cone and the rock cut slope face.



Software: RocScience *Dips*, ver. 5.1

80 degree rock cut slope face angle shown
Variability in rock cut slope alignment shown

Figure 5A – Kinematic Analysis of Wedge Sliding Stability – Unstable Cut Geometry

Figure 5A illustrates three rock cut face alignments, N64°E (Dip Direction 334 degrees), N79°E (Dip Direction 349 degrees) and N92°E (Dip Direction 2 degrees), each having a proposed cut face inclination of 80 degrees. The variability in the rock cut alignment that is shown could represent various side walls of an excavation, or segments along a radius of curvature. This figure clearly illustrates that the orientation of the rock cut face does not govern the potential stability (or instability) of the excavation, as has been illustrated with the toppling and planar sliding failure mechanisms. Rather, the stability of the excavation is controlled by the inclination angle or relative steepness, of the excavation side walls. It is illustrated that a rock cut face angle of 80 degrees meets at the convergence of the planes of Set 3 and Set 1 discontinuities. It is shown that this slope geometry does not provide for a stable configuration, and tetrahedral blocks could dislodge from the rock face as a result of the intersecting discontinuity sets.

When the cut face angle is reduced, as shown in Figure 5B, the potential for wedge sliding stability failure is reduced. The intersection of the planes associated with discontinuity Set 1 and Set 3 no longer converge within the potential wedge sliding zone. This results in a more stable cut orientation.

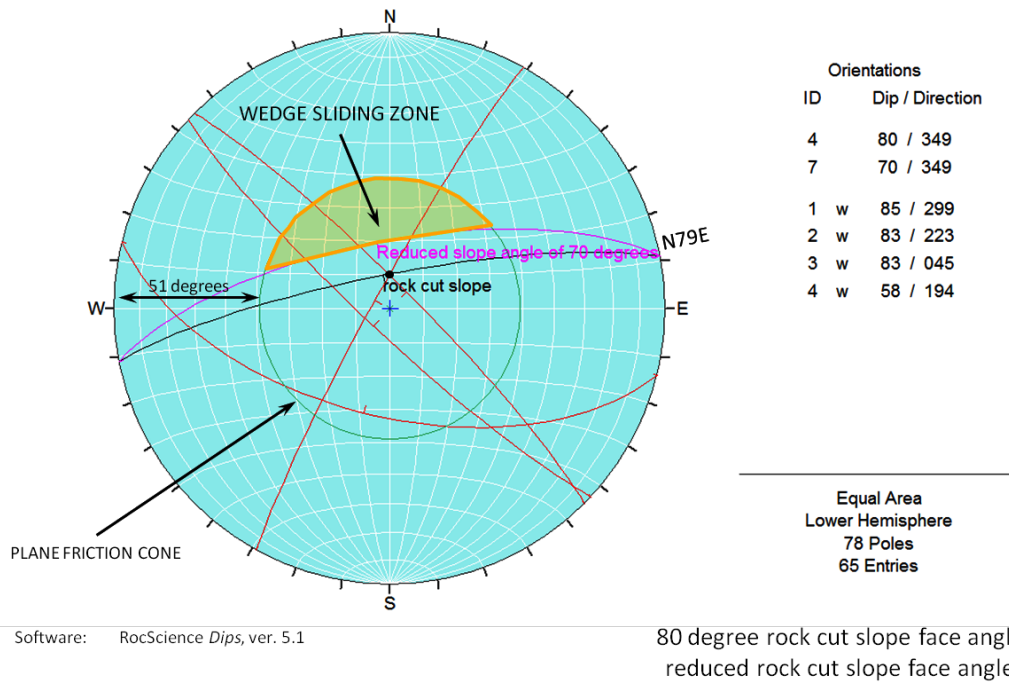


Figure 5B – Kinematic Analysis of Wedge Sliding Stability – Stable Cut Geometry

V PROJECT CASE HISTORY

Excavation and construction adjacent to St. Catherine Siena Church at 411 E. 68th Street in Manhattan. NE- and NW-trending faults and joints were observed to cross-cut the foundation excavation. Unfavorably oriented intersecting geological structural weaknesses along the west wall of this site (dipping into the open excavation) required careful planning to mitigate risk associated with settling and potential failure. NE-trending faults and joints were recognized to be more pervasive along the west wall excavation, thus requiring aggressive rock-bolt support as the excavation progressed. Most of the faults and foliation planes dip southward and out of the west excavation wall, creating a general unfavorable orientation for rock wall stability (Figure 6).

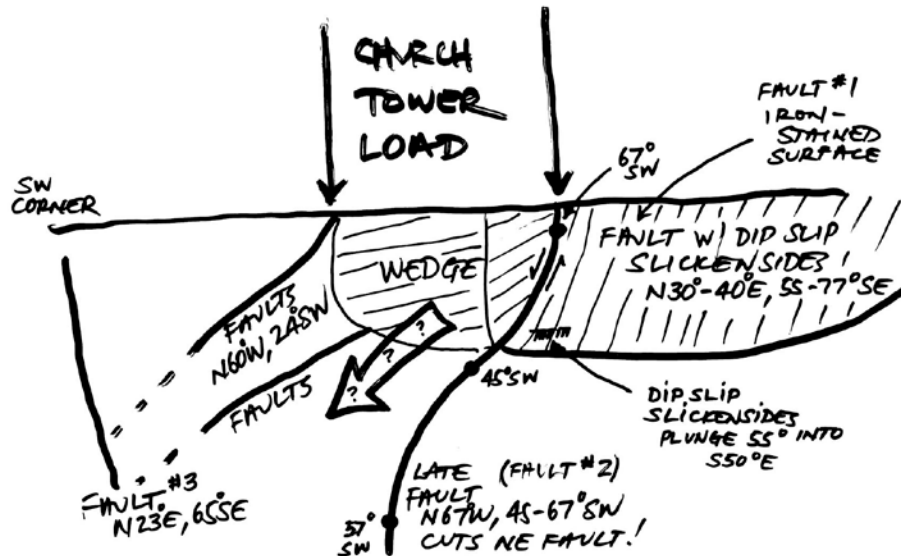


Figure 6 – Field sketch of the west wall of the excavation showing the extent of the structural discontinuities in and around the bedrock wedge perched beneath the church bell tower. Note the low-angle foliation (narrow ruling) in the wedge-shaped block below the church tower.

The primary weakness surfaces of the west wall were associated with the SE-dipping internal layering and foliation and the prominent iron-stained normal “gravity” fault that dips into the excavation from the top of the wall near the church bell tower (the ruled surface in Figure 6; ~N30°-40°E, 55°-77°SE). The steep N-facing iron-stained vertical face of the same wedge is part of a late strike-slip fault (Fault # 2 in Figure 6 = N67°W, 45° to 67°SW).

Many normal faults parallel to Fault #1 (above) were observed along the entire west wall from the SW corner to the NW corner and some conjugate faults were found as well (~N32°E, 68°NW). Thus, the west wall excavation exposed a family of NE-trending faults and associated weakness surfaces parallel to Fault #1 (See Figure 7). In addition, about midway along the west wall, a right-lateral NW-trending strike-slip fault (N75°W, 50°SW) similar to Fault #2 was found to cut the NE-trending fault system. The NE-trending faults were cut by younger NW-trending faults that chop the bedrock into fault-bounded blocks. The deepening of the excavation exposed NE-trending faults more numerous than originally reported along the west wall. The faults were dip-slip, commonly outlined by orange-brown stilbite and whitish calcite mineralization making them readily easy to identify. Most, if not all, of the prominent SE-dipping planar discontinuities exposed along the west wall were faults (see Figure 7).



Figure 7 – Condition of west wall (view facing SW) as seen during excavation. Note the multitude of NE-trending faults along the wall that are parallel to the prominent iron-stained fault (#1 from Figure 6). At least five faults were exposed diagonally from the upper right corner of the digital image to the area beneath the church bell tower.

The wedge of schistose rock under the church tower rests above a prominent normal fault surface with steep dip-slip slickensides. This iron-stained fault surface (Fault #1 in Figure 6) cuts the north wall of the excavation and projects into the SW corner of the site including the area of the church bell tower. At least five NE-trending faults and related discontinuities are visible in the wedge beneath the church bell tower (Figure 8).

The presence of intersecting natural geological structural weaknesses that were unfavorably oriented at this site required aggressive rock bolting and continued vigilance as the excavation continued along the west wall and SW corner. As many of these potential failure surfaces projected upward to intersect the ground surface beneath the church foundation, careful monitoring of the excavation stability was essential to mitigate risk during and after construction.

Figure 8 – View beneath the church tower of the “hanging-wall” wedge of Hartland schistose rock above Fault #1 (upper right corner). Note the prevalence of NE-trending faults and joints parallel to the steep iron-stained failure surface of Fault #1 (at least five surfaces visible in this view) and the NW-trending cross fault (Fault #2 in Figure 6) that curves below the wedge.



VI CLOSURE

The stability of rock excavations is governed by the geology of the rock mass in which the excavation is made. The geologic character of excavations in rock is significantly influenced by the naturally occurring structural geologic features such as the foliation parting and crosscutting discontinuities (shears, faults, and joints). The overall purpose of a geological mapping program is to define the structural features such as joints or faulting, which may control stability of a rock mass. Geotechnical laboratory testing data, in combination with field measurements and numerical methods (e.g., Hoek-Brown failure criterion) may be used to determine rock mass strength characteristics.

These structural geological features that influence rock cut stability often occur in three dimensions with a degree of natural scatter. In order to evaluate stability, it is necessary to analyze the data kinematically using stereographic projection techniques. An equal area stereographic projection of the structural discontinuity data will delineate concentrations of poles that represent preferred orientations, or sets of discontinuities. Toppling, planar sliding and wedge sliding stability analyses may then be performed by kinematic analysis of the stereographic structural data. These analyses demonstrate the powerful use of relatively simple graphical illustration to solve complex engineering geology problems with specific application to foundation excavation. When combined with relatively simple statistical techniques, a probability of failure may be assigned and associated volumes of rock may be estimated.

Owners, design professionals, and contractors must be increasingly aware of potential site constraints and limitations when extending foundation excavations to a project's physical limits. Excavations designed to extend the limits of the site footprint to maximize prime real estate settings and building floor-space often require excavation support or underpinning for adjacent structure foundation elements and infrastructure including utilities, retaining walls supporting sidewalks, roadways and other areas in common rights-of-way.

The structural geology underlying a project's footprint has a direct and controlling factor on efficient and low-risk rock excavation. Stability should be evaluated through comprehensive kinematic analysis in the design phase, in an effort to mitigate potential problems during construction such as excessive over-break and wedge-type fallout, remedial support and underpinning of adjacent structures or costly differing site condition claims.

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