GEOLOGIC IMPACTS ASSESSMENT REPORT

VAN NESS AVENUE BUS RAPID TRANSIT PROJECT

SAN FRANCISCO, CALIFORNIA

Prepared for:

Parsons Transportation Group

Submitted by:

AGS, INC.

JULY 2009

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TABLES

PLATES

1.0 INTRODUCTION

AGS, Inc., (AGS) conducted a Geologic Impacts Assessment of the Van Ness Avenue Bus Rapid Transit (BRT) Project for the San Francisco County Transportation Authority (SFCTA) according to guidelines set forth in the subconsultant agreement between Parsons Transportation Group, Inc., (PTG) and AGS, dated August 1, 2007.

1.1 PROJECT DESCRIPTION

The SFCTA, in cooperation with the Federal Transit Administration (FTA) and the San Francisco Municipal Transportation Agency (SFMTA), proposes to implement BRT improvements along a 2.2 mile stretch of Van Ness Avenue (including a one-block portion of South Van Ness Avenue) in San Francisco, from Mission Street at the south to North Point Street at the north. Van Ness Avenue is one of San Francisco's key north-south arterials that is also designated as U.S. 101, connecting freeway entrances and exits to the south of the City with Lombard Street and the Golden Gate Bridge that provide access north of the City. The Van Ness Avenue BRT Project alignment is shown on **Plate 1**.

Four alternatives have been defined for the proposed Van Ness Avenue BRT Project, including one no-build alternative and three build alternatives. All of the build alternatives include the following elements: a lane dedicated to transit (except for Alternative 2, which would allow shared use for right-turning traffic and parking); higher capacity bus vehicles; level boarding from curb to bus; replacement of the Overhead Contact System (OCS) poles/street lights; sidewalk extension, or bulbs, at corners; pedestrian safety, landscaping and streetscape improvements and amenities; access and lighting improvements; high quality stops/stations; proof of payment/all door boarding/fare prepayment; and, transportation system management (TSM) capabilities.

The build alternatives for the Van Ness Avenue BRT Project would convert either the inside or outside traffic lanes in both the north and southbound directions into dedicated bus lanes. The project improvements would be confined largely within the right-of-way along Van Ness Avenue. The three proposed configurations for the BRT are: (1) a dedicated side bus lane with parallel parking; (2) a dedicated center bus lane with right side boarding platforms and dual medians; and (3) a dedicated center bus lane with left side boarding platforms and a single center median. In order to implement the BRT improvements, there would be accompanying changes to the parking lanes and bus stops along the alignment. Expected project work would include asphalt paving and repairs, wherever necessary; various types of marking and remarking of pavement; construction of concrete ramps, boarding platforms, and pedestrian walkways, as necessary; and installation of bus shelters and signs. A majority of the excavations for these improvements are anticipated to be relatively shallow, to be limited by the weight and foundation types of the planned new structures.

The SFMTA, together with the Public Utilities Commission (PUC), would replace the street lights, which also function as OCS support poles. This construction would be coordinated as part of the build alternatives, and would include removal of existing OCS poles/street lights, and installation of new poles and lights. In most cases, new poles would be installed adjacent to existing poles, approximately 5 to 10 feet to the north or south of the existing poles. Installation of new poles is anticipated to involve excavations up to 13 feet below ground surface (bgs) to accommodate the new pole foundations that are 9.5 to 10 feet in depth and up to 2.5 feet in diameter. Following installation of the new poles and electrical wiring, the original poles and foundations would be removed to approximately 3 feet below street grade, while the remainder of the original pole foundations would be left in place below the ground surface. It is anticipated that in 10 to 20 percent of the cases, the existing pole locations may need to be reused as new pole locations because no other alternatives would be possible. In these cases, once the wire support spans are installed on temporary wood poles located adjacent to the existing poles, the original poles and foundations would be removed in their entirety (except for salvageable attachments) before the new pole foundations would be constructed in the same excavation. Removal of the original pole foundations is anticipated to involve excavations up to 13 feet bgs. Once the new poles and lights are installed and connected, the temporary wooden poles would be removed. Additionally, the deepest excavation work will likely involve installation of new signal poles with excavations to 16 feet bgs.

1.2 MATERIALS REVIEWED

Materials reviewed for this study included: (1) Published and online maps and reports presenting data on regional geology, seismic hazards and faulting; (2) San Francisco City records of geotechnical and environmental site investigations; and (3) Planning and database sources, including the San Francisco General Plan, the Van Ness Avenue Area Plan, and the Environmental Data Resources, Inc., (EDR) database search report prepared for this project.

2.0 EXISTING CONDITIONS

2.1 REGIONAL GEOLOGY

The project alignment, located in San Francisco, California, is situated within the Coast Ranges Geomorphic Province, an active tectonic region characterized by a high level of seismic activity. The Coast Ranges Geomorphic Province includes the northwest trending belt of mountain ranges, valleys and basins that parallel the California coastline from Point Conception northward to the Oregon border. This Province forms a nearly continuous barrier between the Pacific Ocean to the west and the San Joaquin and Sacramento Valleys to the east. The structural depression of San Francisco Bay and the alignment of the ridges and valleys is a consequence of long-term ground deformation resulting from regional tectonic stresses. These stresses are periodically relieved by ruptures occurring along the active fault traces in the region, notably along segments of the San Andreas Fault system and other related faults.

The area east of the San Andreas Fault, including the project alignment, is underlain at depth by late Mesozoic era (Jurassic to Cretaceous) bedrock of the Franciscan Complex, consisting mainly of shale, sandstone, chert, pillow basalt, and serpentinite (Graymer, R.W. et. al., 2006). The bedrock is exposed in erosive cuts, bluffs, and also in the steeper terrain where it has remained uncovered by dune sand, alluvium, or artificial fill. The type of bedrock that is present reflects the tectonic environment in which it formed, ranging from a deep offshore to shallow onshore margin, where sediment was initially being compressed to form rock over the top of the underlying oceanic crust and later deformed in the process of the Pacific Plate being subducted underneath the North American Plate. This type of tectonic regime continued until a shift during the Late Cenozoic Era, between 30 million years ago (Ma) and 25 Ma, when lateral strike-slip motion along the ancestral faults of the San Andreas System became prevalent (Atwater, 1970).

Surficial geology across San Francisco, including the pattern of stream and hillside erosion, alluvial fan and marsh development, and the distribution of dune deposits, have all been strongly influenced by Late Tertiary (Pliocene) and Quaternary (Pleistocene and Holocene) climate and sea-level changes. During the Pliocene, Pleistocene, and Holocene periods, unconsolidated sedimentary deposits that are nearly 400 feet thick accumulated in the San Francisco Bay structural depression. San Francisco Bay itself is a relatively young feature formed from flooding of the Pacific Ocean through the Golden Gate since the start of the Holocene (approximately 11,800 years ago, Helley, E.J., et. al., 1979). The maximum recent extent of the Bay, prior to placement of artificial fill on top of land surrounding the bay, generally corresponds to the mapped extent of the Young Bay Mud Deposit.

2.2 AREAL GEOLOGY

The United States Geological Survey (USGS) recently mapped the geology of the northern San Francisco Peninsula. USGS maps pertaining to the project alignment include the Geologic Map of the Northern San Francisco Quadrangle (Schlocker, 1974), Quaternary Geology and Liquefaction Susceptibility of San Francisco (Witter, R.C. et. al, 2006), and the Geologic Map of the San Francisco Bay Region (Graymer, R.W. et. al., 2006).

In the Civic Center and South of Market areas, deposits of dune sand and alluvium are more than 200 feet thick (Joyner, 1982). The sedimentary deposits thin out on the sides of Nob Hill, Pacific Heights and Russian Hill, including the area of the project alignment, where Franciscan bedrock is likely to be found at moderately shallow depths of less than 100 feet. The geologic maps indicate four (4) distinct units underlie different portions of the project alignment, as shown on **Plate 2**. From youngest to oldest, these units are historic fill, dune sand, alluvium, and Franciscan Complex Bedrock. A description of each of these units follows.

2.2.1 Historic Fill

Piecemeal filling of the Bay and tidal marshlands began in the mid-1800's to provide land for industrial development, and in some cases to aid in general disposal of excavated soils, debris, and rubble, particularly following the 1906 earthquake.

More recently, engineered fill has been placed beneath modern structures and roadways. In addition to earth fill, fill materials include recycled fill materials, such as aggregate base rock, recycled asphaltic pavement, bricks and concrete rubble.

The composition of artificial fill is often highly variable, but commonly consists of a loose to medium dense matrix of clay, silt, sand and gravel with occasional rubble and debris. In central San Francisco, including the area of the project alignment, fill soils are often sandy since they were borrowed from neighboring dune deposits (Helley et. al., 1979). Also included in the mapped areas of artificial fill are small areas of Holocene alluvial deposits, which are too small to be mapped at a city-wide scale (Witter, R.C. et. al, 2006).

2.2.2 Dune Sand

Dune sands are poorly graded, fine- to medium-grained deposits of windblown sand that are typically loose to medium dense and unconsolidated. They cover much of the northern and western areas of San Francisco. While actively shifting sand dunes occupied much of northern and western San Francisco until relatively recently, their development and extent was strongly influenced by recent geologic changes in sea level, particularly during the latest Pleistocene to early Holocene. The aerial exposure of sands, predominantly derived from fluvial and glacially derived sediment from the Sierra Nevada mountains and deposited onto the Continental Shelf by the Sacramento and San Joaquin Rivers, reached its maximum extent during periods of lower sea-level, (Atwater and others, 1977). The aerial exposure combined with the steady prevailing winds along the shoreline contributed to the extent and depth of dune deposits in San

Francisco, including areas of in filled topographic depressions and also at overlapping hillside margins.

2.2.3 Undifferentiated Alluvial Deposits

Alluvial deposits of early to late Pleistocene-age (0.3 to 1.8 Ma) are found at the surface in some of the lower elevation areas and valleys where they have not been covered by later dune sands or fill material. Their composition is more variable than the poorly graded dune sands, and includes gravel, sand, silt, and clay. Included within these undifferentiated alluvial deposits is the Colma Formation, which consists of marine, estuarine and fluvial, fine- to medium-grained sands containing varying amounts of silt and clay, and zones that may be semi-consolidated and weakly cemented (Bonilla, 1971).

Depending upon the age, exposure and mineral constituency of the parent alluvium, some soils horizons may have developed, particularly in response to vegetative cover. Typical soils in the San Francisco climate are alfisols, ultisols, and soils containing a silicic or calcic hardpan (Witter, R.C. et. al, 2006).

2.2.4 Franciscan Complex Rocks – Sandstone and Interbedded Shale

Franciscan Complex bedrock of Cretaceous to late Jurassic-age (115 to 165 Ma) underlies much of the San Francisco Peninsula, east of the San Andreas Fault. It generally consists of highly deformed, altered and fractured volcanic, sedimentary, and metamorphic rock, but also has some relatively intact blocks of only lightly deformed or metamorphosed rock.

Common Franciscan rock types include sandstone, shale, chert, basalt, and serpentine. Near the project alignment, the Franciscan bedrock is exposed on Russian Hill, Nob Hill and Pacific Heights, where sandstone and interbedded shale with minor conglomerate layers are common (Schlocker et. al., 1974). On the project alignment, Franciscan bedrock is mapped at shallow depths beneath Van Ness Avenue on the side of Russian Hill, between the intersections of Greenwich Street and Lombard Street. In the area of Van Ness Avenue, north of O'Farrell Street, the Franciscan bedrock is typically overlain by 30 to 200 feet of alluvium and dune sands, increasing to a thickness of 200 feet or more in the area south of O'Farrell Street to Mission Street (Joyner et. al., 1982).

2.3 TOPOGRAPHY

The terrain in the project area of San Francisco is characteristically hilly, consisting of gentle to moderately-steep sloping ridgelines or hills and spur ridges ranging from an elevation of 200 feet up to over 900 feet, which are separated by small valleys or basins. The project alignment crosses near the low point of one of these east-west trending ridgelines that connects Nob Hill to the east and Pacific Heights to the west. Further north, the project alignment crosses near the western toe of Russian Hill.

Local variations in slope reflect the drainage pattern, with erosion having been more prevalent during the Pleistocene, when sea levels were often nearly 300-feet lower. The valleys and basins were typically filled by sediments, particularly by the irregular forms of alluvium and dune sands. To a lesser extent, the native topography has been altered by urban development, particularly by the grading and placement of fill materials to varying extents along the entire length of the project alignment.

2.3.1 Mission Street to McAllister Street

This approximately 2,600-foot long segment of Van Ness Avenue, located between Mission and McAllister Streets, ascends a gradual southeasterly facing slope at a gradient of less than 1.5 percent. Ground elevations are approximately 44 feet above mean sea level at Mission Street, 55 feet at the Market Street intersection, and 74 feet at the McAllister Street intersection.

2.3.2 McAllister Street to Clay Street

The topography along this approximately 4,490-foot segment is characterized by a gentle south-facing slope with a gradient of between 2 and 5.5 percent, reaching the crest of the hill near the California Street intersection. Between California Street and Clay Street, the gradient is nearly level (less than 1.5 percent). The maximum elevation of nearly 200 feet occurs at the Clay Street intersection.

2.3.3 Clay Street to Union Street

The topography along this 2,320-foot long segment is characterized by a north-facing slope with a gentle to moderate gradient of between 2 to 8 percent. The steepest slopes exist between the Pacific Street and Broadway Street intersections (8 percent), and between the Broadway and Vallejo Street intersections (6.5 percent). Elevations range from approximately 99 feet near the Union Street intersection to 200 feet at Clay Street.

2.3.4 Union Street to North Point Street

The topography along this 1,060-foot segment is characterized by nearly level ground with less than 1 percent slope between Union and Filbert Streets, a short south facing slope between Filbert and Greenwich Streets of 3 to 4 percent, and a short north facing slope between Greenwich and North Point Streets of 3 to 4 percent. Elevations range from 99 feet at both Filbert and North Point Streets, to 110 feet at the Greenwich Street intersection.

2.4 SUBSURFACE SOIL CONDITIONS

The subsurface soil conditions underlying the proposed project alignment were evaluated by reviewing available maps and publications, and geotechnical investigation reports for buildings and structures in the project vicinity, typically less than 1,000 feet from Van Ness Avenue. The geotechnical investigation reports reviewed were obtained through the San Francisco Department of Building Inspections.

2.4.1 Mission Street to McAllister Street

As shown on **Plate 2**, dune sand (Qds) is mapped underneath this segment of the project alignment. Underneath the dune sand are variably thick layers of older alluvium and at depth, Franciscan Complex bedrock. Overlying the dune sand are local areas of historical fill, including pavement fill and structural fill underneath the buildings and structures (Witter, R.C. et. al., 2006).

The available subsurface information for this segment is derived from a geotechnical investigation report completed for a seismic upgrade at 30 Van Ness Avenue, near the Hickory Street intersection (Treadwell and Rollo, 1997). According to this report, there is approximately 6 to 8 feet of loose to medium dense sandy fill material at the site. Beneath the sandy fill soils, medium to very dense sand exists to a depth of 25 to 30 feet bgs. From nearly 30 to 40 feet bgs loose to medium dense clayey sand exists (possibly an old buried soils horizon or paleosol). Groundwater was encountered at a depth of approximately 20 feet. For deeper subsurface information the Treadwell and Rollo report cited the investigation completed for the Van Ness Avenue MUNI Station in the area of the Market Street intersection. Logs from the subsurface exploration completed for the MUNI station indicate very dense clayey sand or stiff sandy clay of the Colma Formation that exist in the area from about 40 feet to as deep as 125 feet, which is the deepest exploration depth.

2.4.2 McAllister Street to Clay Street

Dune sand (Qds) is also mapped underneath this segment of the project alignment, but the depth to bedrock is expected to be shallower than further south, particularly at the higher elevations between California and Clay Street (Joyner, 1982).

Globe Soils Engineers (1998) completed a geotechnical investigation report for the property at 900 Van Ness Avenue, near the Ellis Street intersection. In that investigation they drilled two borings less than 100 feet east of Van Ness Avenue. In Boring EB-1, sandy loose fill with rock fragments was penetrated to a depth of 5 feet bgs. Loose silty sand was penetrated from 5 to 10 feet, while medium dense sand (dune sand) was found to continue from 10 to 15 feet. Dense sand, including some well-graded sand (alluvium) was penetrated from 15 to 20 feet, while dense, silty, fine-grained sand was penetrated from 20 feet to the bottom of the hole at a depth of 25 feet. Similar conditions were encountered in Boring EB-2, except the boring only penetrated to a depth of 20 feet. Groundwater was not encountered in either boring.

Geophysical data from the site indicates average conditions across the site consisting of 4 feet of loose fills soils (characteristic velocity of 2400 feet per second (fps)), 10 feet of stiff soils and sand (characteristic velocity of 3500 fps), and hard sediments (characteristic velocity of 5100 fps) deeper than 14 or 15 feet bgs (Globe, 1998).

Cooper Clark and Associates (1976) completed a geotechnical investigation report for the property at 1595 Van Ness Avenue, near the California Street intersection. After coring through a 6-inch thick concrete floor slab, a fine-grained, medium dense sand was encountered to a depth of 10 feet, followed by dense sand from 10 to 20 feet. The sand was reported to grade to a firm sandy clay at 22 feet. Clay, with occasional sandy lenses, continued to a depth of 39 feet. Dense clayey sand was penetrated from 39 feet to the maximum depth explored of 50 feet bgs. Groundwater was not encountered.

2.4.3 Clay Street to Union Street

Dune sand is mapped as far north as the Broadway Street intersection. A large contiguous deposit of fill is mapped north of the Broadway Street intersection, to the south of the Union Street intersection. Immediately south of the Union Street intersection there is a contact between the fill to the south and native alluvial soils to the north (Witter, R.C., et. al., 2006).

According to a geotechnical investigation report conducted for the property at 1401 Broadway, near the intersection of Larkin and Broadway (two blocks east of Van Ness Avenue), six borings were drilled to as deep as 26.5 feet bgs (ERRG, 2003). In the uppermost 23 to 24.5 feet, medium dense poorly graded sand or silty sand was encountered, with a few lenses of gravel. Silty clay was found beneath the sand, which was described as medium-stiff to stiff. Groundwater was not encountered.

2.4.4 Union Street to North Point Street

Alluvium is mapped underneath the Union Street intersection northward to the western portion of the Greenwich Street intersection, where there is a contact with the underlying Franciscan sandstone and shale bedrock. Shallow bedrock occurs beneath the eastern portion of the Greenwich Street intersection northward to the southern edge of the Lombard Street intersection. Alluvium is mapped underneath the actual Lombard Street intersection northward to the North Point intersection (Graymer, R.W., et. al., 2006).

According to a geotechnical investigation report conducted for the property at 2433 Larkin Street, between Greenwich and Filbert Streets (two blocks east of Van Ness Avenue), logs of test pits excavated at both 1271 and 1269 Lombard Street indicate that shallow rock exists in the area since sandstone and shale rock was penetrated at a depth less than 5 feet beneath the ground surface (Earth Mechanics, 2003). While major groundwater was not reported, seepage was seen at the contact between the overlying soil and the underlying weathered rock.

2.5 GROUNDWATER

California Groundwater Bulletin 118 (http://www.groundwater.water.ca.gov/bulletin118/) indicates that the project area includes portions of the downtown groundwater basin (basin number 2-40). The basin is bounded to the west and northwest by the Twin Peaks Ridgeline, and includes the Nob Hill and Telegraph Hill areas to the north, the Potrero Point area to the east, and most of the downtown area. The average annual precipitation within the basin is approximately 24 inches. The primary water-bearing formations are comprised of unconsolidated sediments that include alluvial fan deposits, beach and dune sands, undifferentiated alluvium, and artificial fill. None of the geologic formations along the project alignment are considered useful aquifers due to poor overall water quality and high concentrations of undesirable minerals.

Geologic mapping indicates that the groundwater table occurs less than 20 feet bgs in most of the lower lying areas along the project alignment, where the ground elevation is less than approximately 150 feet above mean sea-level (Knudsen et. al., 2006).

Monitoring well data provided in the EDR database report (EDR, 2008) indicates a depth to groundwater ranging from 5 to 20 feet bgs is common in two areas; (1) along Van Ness Avenue from Mission Street northward to the vicinity of the Geary Boulevard intersection; and (2) North of the Broadway intersection to Lombard Street. Between Geary Boulevard and the Broadway intersection the monitoring well data indicates either no groundwater was encountered, or that depths to water exceed 20 feet. In general, the reported groundwater levels are only representative of the conditions at the time of drilling and/or monitoring well measurement, and are expected to vary both seasonally and annually based on the rainfall pattern, microtopography and distribution of impervious surfaces, and where present, the pattern of groundwater withdrawal or localized pumping. Urban sources of groundwater (in addition to normal infiltration of rainfall) include trapped pipe and culvert leakage, and irrigation runoff. Where subsurface drainage is obstructed in the urban environment it is possible that shallow pockets of groundwater may persist for weeks and even months after the last substantial rainfall.

The direction in which groundwater flows varies with the topography. The EDR Geocheck™ Report (2008) indicates groundwater flow in the vicinity of Mission and Market Streets is to the east, on the south facing hillside north of the Civic Center the flow is generally to the south or southeast, and on the north facing hillside north of Clay Street flow is generally to the northwest.

2.6 FAULTS AND SEISMICITY

2.6.1 Historical Seismicity

The project alignment is located within a seismically active region, subject to major earthquakes capable of producing strong to violent ground shaking. While no active faults are known to cross the project alignment, several major active faults are mapped within 30 miles, including the San Andreas, Hayward, Calaveras, and San Gregorio Faults. Regional historical earthquakes are summarized in **Table 1**, Historical Earthquakes.

TABLE 1
HISTORICAL EARTHQUAKES

Date	Magnitude	Fault	Epicenter Area
June 10, 1836	6.5[1], 6.8[5]	San Andreas	San Juan Bautista
June 1838	7.5[1], 7.0[5]	San Andreas	San Juan Bautista
Nov. 26, 1858	6.25[5]	Calaveras	San Jose Area
October 8, 1865	6.3[2], 6.5[5]	San Andreas	South Santa Cruz Mountains
October 21, 1868	7.0[2,5]	Hayward	Berkeley Hills, San Leandro
February 17, 1870	6.0[5]	San Andreas	Los Gatos
April 19, 1892	6.5[5]	Uncertain	Vacaville
April 21, 1892	6.25[5]	Uncertain	Winters
June 20, 1897	6.25[5]	Calaveras	Gilroy
March 31, 1898	6.5[5]	Uncertain	Mare Island
May 19, 1889	6.25[5]	Uncertain	Antioch
April 18, 1906	7.9[3]	San Andreas	Golden Gate
July 1, 1911	6.6[4], 6.5[5]	Calaveras	Diablo Range, East of San Jose
October 22, 1926	6.1[5]	San Gregorio?	Monterey Bay
April 24, 1984	6.1[5]	Calaveras	Morgan Hill
October 17, 1989	7.1[5]	San Andreas	Loma Prieta, Santa Cruz Mountains

(1) Borchardt & Toppozada (1996)
(2) Toppozada et al (1981)
(3) Petersen (1996)
(4) Real et al (1978), Toppozada (1984)
(5) Ellsworth, W.L. (1989)

2.6.2 Active Faults

When accumulated strain within the crust of the earth is released by slip along a fault, the subsequent release of seismic energy and ground motion is known as an earthquake. The locations of the major active faults in the area, the most likely seismic sources, are shown on **Plate 3**. Fault characteristics of the major active faults located less than 30 miles from the project alignment are presented in **Table 2.**

Very strong or even violent ground shaking would likely occur in response to a maximum moment magnitude earthquake on the San Andreas Fault, located approximately 6.8 miles southwest of the project alignment. Strong ground shaking is likely to occur in response to a maximum moment magnitude earthquake on the San Gregorio or Hayward Faults, located at respective distances of 10.5 miles southwest, and 11 miles east of the project alignment (Association of Bay Area Governments, ABAG, 2008). Moment magnitude is determined from the physical size (area) of the rupture of the fault plane, the amount of horizontal and/or vertical displacement along the fault plane, and the resistance of the rock type along the fault to rupture. For the major Bay Area faults, each fault is commonly divided into segments, each characterized by a certain slip rate and time activity (USGS, 2008, WGCEP, 2008). These values are also shown in **Table 2**.

TABLE 2
ACTIVE FAULT SEISMICITY

Fault Name	Distance to Project Alignment (miles/kilometers)	Maximum Moment Magnitude[2]	Contributing Segments	Slip Rate[2] (mm/year)
San Andreas	6.8/ 11	7.9	Offshore (SAO), North Coast (SAN), Peninsula (SAP), Santa Cruz Mountains (SAS)	24 ± 3, 24 ± 3, 17 ± 4, 17 ± 4
San Gregorio	10.5/ 17	7.3	Northern (SGN), Southern (SGS)	7 ± 3, 3 ± 2
Hayward	11/ 18	7.1	Northern (HN), Southern (HS)	9 ± 2
Calaveras	23/ 37	6.8	Northern (CN), Central (CC), Southern (CS)	6 ± 2, 15 ± 3, 15 ± 3
Concord-Green Valley	25/ 40	6.9	Concord (CCD), Green Valley (GV)	4 ± 2
Rodgers Creek	28/ 45	7.0	Rodgers Creek (RC)	9 ± 2
West Napa	29/ 47	6.7	West Napa (WN)	1 ± 1
Greenville	29/ 47	7.0	Northern (GN), Southern (GS)	2± 1

1. Jennings (1992)
2. WGCEP (2008), Working Group on California Earthquake Probabilities, Ellsworth Magnitude.
3. USGS (2008), National Seismic Hazard Report.

2.6.3 Maximum Capable Earthquake

The M_{max} earthquake is the largest earthquake that a given fault is considered capable of generating. For the project alignment, the controlling M_{max} earthquake based on moment magnitude would be a magnitude 7.9 event occurring on the San Andreas Fault, located approximately 6.8 miles (11 km) to the southwest of the southern end of the project alignment.

Correlations between distance from a causative fault and mean values of the peak bedrock accelerations and the effects of local soil conditions on peak ground accelerations have been developed by Abrahamson and Silva (1997), Campbell and Bozorgnia (2003), Boore, Joyner and Fumal (1997), and Sadigh (1997). Based on these correlations for an M_{max} 7.9 event occurring on the San Andreas Fault, the mean peak ground surface acceleration for the project alignment is estimated to be nearly 0.60 g. The California Geological Survey estimates peak ground acceleration between 0.50 and 0.60 g for the 10 percent in 50-year probabilistic event.

For an M_{max} 7.1 earthquake occurring on the Hayward Fault, the mean peak ground surface acceleration within the project area is estimated to range from 0.30 to 0.40 g (ABAG, 2008).

3.0 SEISMIC HAZARDS

Structures may be damaged and people may be injured or killed as a result of both the primary and secondary effects from earthquakes (seismic hazards). Seismic hazards include fault rupture, ground shaking, ground settlement, liquefaction, landslides, and tsunamis. The potential for these hazards to occur, applicable to the project alignment in San Francisco, is discussed in this section.

3.1 FAULT RUPTURE

Fault rupture could occur anywhere, but will probably occur either along or close to the trace of currently active faults. There is no Alquist Priolo Earthquake Fault Zone Map covering the San Francisco North Quadrangle (CGS, online 2008), which includes the area of the project alignment, and geotechnical investigation reports completed in the area did not identify faulting. Since there is no evidence to indicate that the project alignment crosses an active fault, impacts due to surface fault rupture from a future earthquake are considered unlikely and no mitigation is proposed.

3.2 GROUND SHAKING

As discussed in Section 2.6.2, very strong ground shaking is considered quite possible. The severity of future ground shaking along the project alignment will be influenced by a number of factors, including the proximity of the project alignment to the location of the causative earthquake, the duration and intensity of the earthquake, and the type of geologic materials underlying the site. Amplification of seismic waves is possible in loose or soft soils, while seismic waves should attenuate or dampen when passing through rock or very hard soils. High amplitude and long duration seismic waves are of concern as studies indicate these types of waves are most likely to produce structural damage.

The mean peak ground surface acceleration for the controlling maximum capable earthquake along the project alignment is computed to range from 0.50 to 0.60 g. This does not include seismic amplification due to any soft or loose soils. The maximum expected intensity of shaking is likely to be similar to that which was experienced during the 1906 earthquake, and is expected to occur infrequently (once per century or less). Moderate intensity ground shaking, with peak ground surface accelerations in the range of 0.30 to 0.50 g, is likely to occur more frequently in response to a major earthquake of magnitude 6.0 or greater occurring on any one of the other large active faults in the area (ABAG, 2008).

An earthquake of similar magnitude to the 1906 earthquake on the San Andreas Fault is expected to cause very strong ground shaking and result in moderate damage, equivalent to a Modified Mercalli Intensity (MMI) of VIII. Moderate damage could include the development of major cracks in the pavement and seismically-induced settlements in some cases, particularly wherever loose fill soils are present. Should underground pipes burst, sinkholes may develop that could damage shallow foundation structures or cause sections of pavement to collapse. Structural damage could occur to weak structures, particularly any unreinforced masonry buildings, whereby bricks, stone, or glass may fall onto the ground (ABAG, 2008).

Although the 2007 California Building Code (CBC) (Based on the 2006 International Building Code, IBC) provides building standards that are designed to prevent building collapse, moderate structural damage could still occur. Compliance with CBC standards would minimize the risk of injury and damage from ground shaking. Furthermore, site-specific seismic design criteria should be developed for all critical structures, and where applicable, for pavement design.

3.3 LIQUEFACTION

Soil liquefaction is a phenomenon in which saturated, cohesionless soils lose their strength due to the build-up of excess pore water pressure, especially during cyclic loadings (shaking) such as those induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if not confined. Soils most susceptible to liquefaction are loose, clean, uniformly graded, fine-grained sands. Gravels and coarse-grained sands are also susceptible to liquefaction, as are saturated silty and clayey sands.

The consequences of liquefaction could easily include seismically-induced settlements, additional lateral loads on piles, down drag forces on pile foundations, localized lateral deformation of soils, and flotation (buoyancy) of underground structures (i.e., tanks, pipelines and manholes) underlain by the potentially liquefiable soils.

Areas considered susceptible to liquefaction are shown on **Plate 4**, which is based on the Seismic Hazard Map of San Francisco compiled by the California Division of Mines and Geology (CDMG, 2000, now the California Geological Survey). Two separate areas of the project alignment are considered susceptible to liquefaction. These are (1) the area between the Union Street and Broadway Street intersections, which is an area where historic fill is mapped; and (2) the area between the Hayes Street and Mission Street intersections, another area where artificial fill is mapped. Other portions of the project alignment are considered to have low to moderate susceptibility to liquefaction.

In general, identification and evaluation of the liquefaction potential should be considered in the geotechnical report. Should the magnitude of the problem be considered unacceptable, the project geotechnical engineer should propose specific mitigations. Below is a discussion of alternative ground improvement techniques that may mitigate the problem. Selection of the appropriate mitigation measure(s) to be used should consider the condition and details of existing structures in which disturbance should be avoided, and also the project scope and constraints.

a) Ground improvement through displacement or compaction grouting. Grouting involves the use of low slump, mortar-type grout pumped under pressure to densify loose soils by displacement.

b) Ground improvement through vibro-compaction or vibro-replacement. These techniques use similar equipment, but different backfill materials to achieve densification of soils at depth. In vibro-compaction, a sand backfill is generally used, whereas in vibro-replacement, stone is used as backfill material. Vibro-compaction is generally effective if the soils to be densified are sands containing less than approximately 10 percent fine-grained material passing the No. 200 sieve. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines.

c) Permeation grouting involves the injection of low viscosity liquid grout into the pore spaces of granular soils. The base material is typically sodium silicate or microfine cements where the D15 of the soil (particle diameter for which 15 percent by weight of the soils are larger diameter) should be greater than 25 times the D85 of the grout for permeation (particle diameter for which 85 percent by weight of the soils are larger diameter).

d) Jet Grouting involves the rapid pumping of grout through a rod inserted into the ground. Grout is jetted outward into the ground through horizontal nozzle(s) in the monitor at a high velocity [typically 650 ft/sec (200m/sec)]. Unlike other methods, the scouring action due to the jetting of grout breaks down the soil matrix and replaces it with a mixture of grout slurry and in situ soil (soilcrete). Depending upon the purpose, grout may be introduced surgically or into a large area. The method is considered one of the most versatile techniques of ground improvement. Single fluid jet grouting is most effective in cohesionless soils, while double or triple jet grouting is more effective in cohesive soils. Jet Grouting is effective across the widest range of soil types, of any grouting system, including silts and some clay. Since jet grouting is an erosion-based system, soil erodability plays a major role in predicting geometry, quality and production.

e) Soil Mixing, also known as the Deep Mixing Method, is the mechanical blending of the in situ soil with cementitious materials (reagent binder) using a hollow stem auger and paddle arrangement. The intent of the soil-mixing program is to achieve improved character, generally a design compressive strength or shear strength and/or permeability.

Where necessary, these mitigation measures should be implemented along with adequate subsurface drainage through use of wick drains or other suitable means. Reducing the volume of groundwater reduces the likelihood of liquefaction occurring in the zone where movement would be most damaging to shallow foundation structures and pavements.

3.4 SEISMICALLY-INDUCED SETTLEMENTS

In addition to liquefaction of saturated soils, seismic shaking may cause settlement of non-saturated soils to occur. Collapse of void space in porous soils would reduce ground volume in a process sometimes called seismic densification. Based on our review of the available data, soils along the proposed alignment seem suitable for support of the light structures that are proposed as part of the project, but there will be some settlement. Seismically-induced settlements are expected to be concentrated where there are loose sandy soils with little fines and high porosity (such as dune sand areas within the project alignment), and also in any unconsolidated fill soils. At a minimum, in response to seismic shaking, consolidation of any previously unconsolidated fill could trigger several inches of ground settlement. Suitable foundation design, including support of structures on dense native soils or engineered fill (never on unconsolidated artificial fill) would reduce future settlement in response to seismic shaking.

Damage to structures and pavements resulting from seismically-induced settlements or instability of subsurface materials is discussed in Section 4.2.

3.5 SEISMICALLY-INDUCED LANDSLIDES

The project area is not considered susceptible to seismically-induced landslides (CGS, 2000). Therefore, no mitigations are proposed.

3.6 TSUNAMIS

A tsunami is a series of traveling ocean waves of extremely long length generated by disturbances associated primarily with earthquakes occurring below or near the ocean floor. Underwater volcanic eruptions and landslides can also generate tsunamis. Tsunamis are a threat to life and property to anyone living at lower lying areas near the ocean. Large tsunamis have been known to rise over 100 feet, while tsunamis 10 to 20 feet high can still be very destructive and cause many deaths and injuries.

The ABAG tsunami evacuation planning maps for the ocean side of San Francisco and San Mateo Counties are based on modeling of potential earthquake sources and hypothetical extreme undersea, near-shore landslide sources. Maximum run-up to a specific contour of 12.8 meters (42 feet) in these two counties was determined to be reasonable. According to the ABAG tsunami evacuation planning map for San Francisco and San Mateo Counties, the project alignment is not located within a tsunami evacuation area.

4.0 OTHER GEOLOGIC HAZARDS

Other types of geologic hazards typically depend upon the ground configuration and stability of underlying materials. These hazards exist regardless of the occurrence of earthquakes, but are affected by factors such as weather and flooding potential, ground loading, construction-induced ground movements, and from other types of natural disasters such as volcanic eruptions, non-seismically generated waves, and the various types of slope failures. Hazards applicable to the project alignment are discussed in the following section.

4.1 SLOPE INSTABILITY

Areas with the greatest potential for slope failure possess steep slopes and weak underlying rock or soils conditions. Increasing the risk of slope failure are saturated ground, rock bedding parallel to the slope gradient, and the occurrence of past landslides subject to reactivation, where there may be a zone or plane of weakness in the subsurface upon which ground movement could be triggered.

For the project alignment the overall risk is limited since slopes are flatter than 10 percent. The steepest slopes are between Pacific and Broadway (8 percent), and between Broadway and Vallejo (6.5 percent). Therefore, a major landslide or slope failure is not likely to occur. There are also no mapped landslides crossing the project alignment (Knudsen, 2000). More likely to occur is minor slope failure; including instability resulting from local construction-induced settlements, or slumping if there were to be an improperly supported excavation near the base of a hillside.

Slope instability may be mitigated by several alternatives, such as flattening the slope and/or unloading the top of the slope, improving drainage, construction of retaining structures near the toe, and soil material improvement. Shoring design of open excavations should be completed in consideration of the surcharge load from nearby structures, including an examination of the potential for lateral movement of the excavation walls as a result. Based on the data underpinning requirements should be established, where necessary. Safe slope angles for cut and fill slopes should also be established depending upon the nature of excavation and filling work that is planned.

4.2 DAMAGE FROM SETTLEMENTS OR INSTABILITY OF SUBSURFACE MATERIALS

Based on our review of the available data, soils along the proposed alignment seem suitable for support of the light structures that are proposed as part of the project, but there will be some settlement. The largest component of future settlement is expected to be seismically induced (See Section 3.3). General settlement is expected to be minor and mainly limited to areas of fill. Fill soils are considered more susceptible to settlement than native soils because of a lack of consolidation over time, especially if fill was improperly compacted. There is also some risk of differential settlement at fill boundaries should the fill soils settle disproportionately with respect to the adjoining native ground.

If left unchecked, settlement could cause damage to structures, cracking of asphalt pavements, the trapping of water from rain, and the deterioration of roadway pavements. Concerning general settlement risk, site-specific geotechnical data should be obtained to evaluate and verify the compressibility and settlement potential of subsurface soils encountered within the project corridor. Settlements can be mitigated by such methods as pre-loading, deep foundations, and soil improvements. Soil improvements intended to mitigate the liquefaction hazard are discussed in Section 3.3, which could also be used to mitigate the hazard from general settlement and ground instability. Other possible foundation design mitigation measures are discussed below.

For the lightweight structures that are proposed, such as the bus shelters, and where structures would be supported on native soils, shallow foundation systems may be used to support the weight of the various structures that are proposed to be built. However, wherever structures are underlain by artificial fill soils, those structures should be supported on deep foundations, unless those fill soils are first over excavated and replaced with engineered fill.

Avoidance of potential settlement may also be achieved by spanning the area with the settlement problem. Spanning structures should be supported on deep foundations extending below any compressible soils. Design of deep foundations must take into consideration down drag loads induced by consolidation due to the weight of adjacent fills. This method involves removal of any compressible soils either by excavation and recompaction, or by displacement. Because excavation and backfilling of compressible soils may be increasingly expensive with depth, the feasibility of excavation and recompaction is limited by the thickness and extent of the compressible soils.

5.0 CONSTRUCTION CONSIDERATIONS

A majority of the excavations are anticipated to be relatively shallow to incorporate footings extending to approximately 2 feet bgs. The deepest excavations would most likely be at the locations of new signal poles with excavations to 16 feet bgs. Additionally, where the existing OCS pole locations may need to be reused as new pole locations, because no other alternatives would be possible, removal of the original pole foundations and installation of new pole foundations is anticipated to involve excavations up to 13 feet bgs.

Based on the review of the available subsurface information and professional judgment, the excavations extending to the proposed depths are anticipated to be made using conventional earthmoving equipment. The excavations must comply with the current requirements of OSHA or Cal-OSHA, as applicable. Additionally, all cuts deeper than 5 feet should be sloped or shored. In areas with space limitations, the excavations will probably need to be shored; however, shallow excavations above the groundwater level may be sloped if space permits. Temporary excavations may be sloped at a horizontal (H) to vertical (V) ratio of 1.5(H):1(V) or flatter above the groundwater level, which is estimated to be as shallow as 5 feet bgs in some areas, depending on the time of year; however, it is the responsibility of the contractor to maintain safe and stable slopes and provide shoring as required during construction. Flatter slopes will be required if clean or loose, sandy soils are encountered along the slope face. Steeper cuts may be utilized for excavations less than 5 feet deep depending on the strength and homogeneity of the soils as observed in the field.

Heavy construction equipment, building materials, excavated soil, and vehicle traffic should be kept away from the edge of the excavation, generally a distance equal to, or greater than, the depth of the excavation.

During wet weather, runoff water should be prevented from entering the excavation, and collected and disposed of, outside the construction limits. To prevent runoff from entering the excavation, a perimeter berm may be constructed at the top of the slope. In addition, it is recommended that the sidewalls of the excavation be covered by plastic sheeting to prevent saturation of the earth material.

During excavations adjacent to existing structures, care should be taken to adequately support facilities that might be affected by the proposed construction procedures. Similarly, the sidewalks, slabs, pavements, and utilities adjacent to the proposed excavations should be adequately supported during construction.

6.0 CLOSURE

This Geologic Hazards Impact Assessment was prepared for the exclusive use of the Parsons Transportation Group, Inc. and its consultants for the specific application to the Van Ness Avenue BRT Project in the City and County of San Francisco, California. This report was prepared in accordance with generally accepted professional geotechnical engineering practice. No other warranty, expressed or implied, is made.

The analyses and recommendations submitted in this report are based upon available data obtained from borings drilled by others and the geologic reports in the site vicinity. No site-specific subsurface data were obtained for this study. The conclusions and recommendations presented in this report are preliminary and should be further verified by site-specific final geotechnical studies. If changed conditions are encountered in the final geotechnical studies, it will be necessary to reevaluate the recommendations of this report.

Respectfully submitted,
AGS, Inc.

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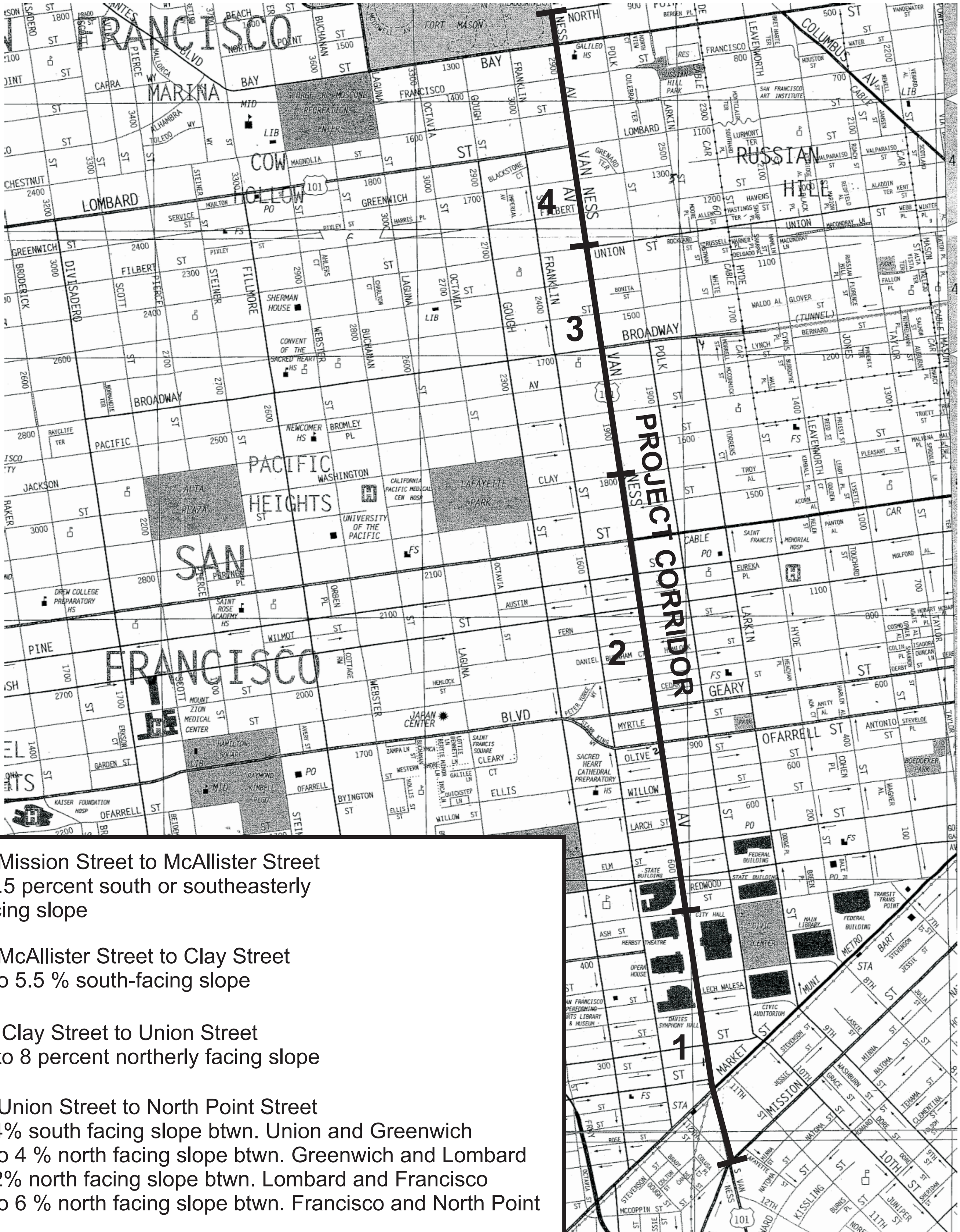
PLATES



1: Mission Street to McAllister Street
<1.5 percent south or southeasterly facing slope

2: McAllister Street to Clay Street
2 to 5.5 % south-facing slope

3: Clay Street to Union Street
2 to 8 percent northerly facing slope

4: Union Street to North Point Street
< 4% south facing slope btwn. Union and Greenwich
3 to 4 % north facing slope btwn. Greenwich and Lombard
< 2% north facing slope btwn. Lombard and Francisco
5 to 6 % north facing slope btwn. Francisco and North Point

0 500 1000 ft

PROJECT SITE / SLOPE MAP

VAN NESS AVENUE BRT EIR/EIS

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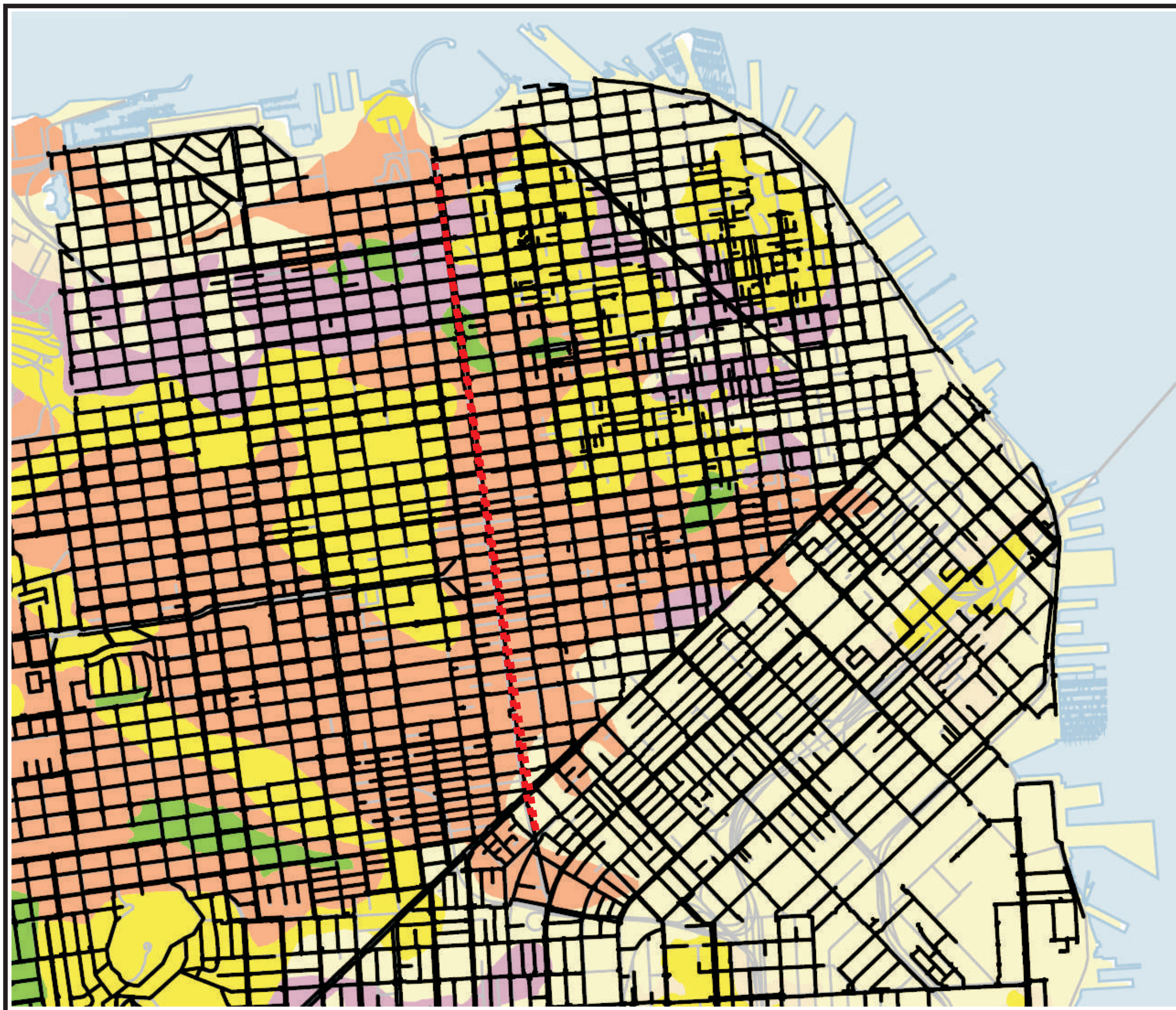
CONSULTING ENGINEERS

JOB NO. KG0205	DATE: JULY 2008	PLATE 1



	afbm	artificial fill over Bay Mud		Qoa	Early to Late Pleistocene alluvial deposits, undifferentiated
	af	artificial fill		Qds	Latest Pleistocene to Holocene dune sand
	Qa	Latest Pleistocene to Holocene alluvial deposits, undifferentiated		br	Early Quaternary and older deposits and bedrock

artificial fill (< 150 years), Holocene-age (<11,800 years), Latest Pleistocene to Holocene-age (<30,000 years)
Early to Late Pleistocene-age (>30,000 years to 1.8 million years), Early Quaternary and older (>1.4 million years)

0 1 mile 0 1km

Project Corridor

SOURCE: Witter, Robert C., Knudsen, Keith L., Sowers, Janet M., Wentworth, Carl M., Koehler, Richard D., and Randolph, Carolyn E., Map of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, USGS Open File Report 2006-1037.

QUATERNARY GEOLOGIC MAP

VAN NESS AVENUE BRT EIR/EIS

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JOB NO. KG0205 | DATE: JULY 2008 | PLATE 2

FAULT RECENCY CLASSIFICATION

FAULTS WITH HISTORIC SURFACE RUPTURE (~180 YA).

FAULTS THAT DISPLACE HOLOCENE (~11 kA) OR LATEST PLEISTOCENE (~20 kA) DEPOSITS OR GEOMORPHIC SURFACES

FAULTS THAT DISPALCE LATE QUATERNARY (~780 kA) DEPOSITS OR GEOMORPHIC SURFACES

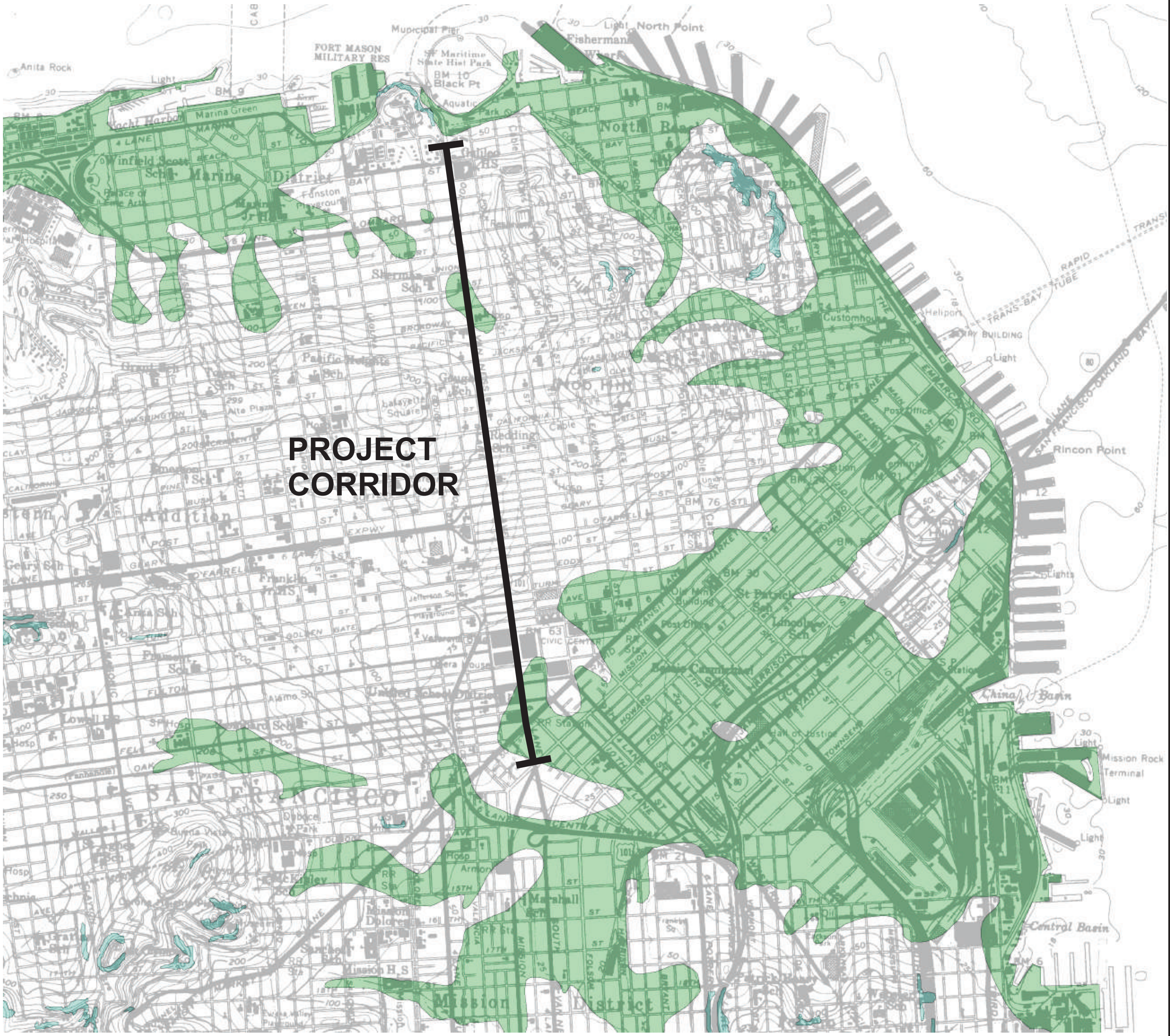
QUATERNARY FAULTS (1.8 MA)

0 10 miles

0 10 km

Base Map: GoogleEarth,2007.
Source: Quaternary fault and fold database of the United States.

EARTHQUAKE FAULT MAP VAN NESS AVENUE BRT EIR/EIS		AGS, Inc. CONSULTING ENGINEERS
JOB NO. KG0205	DATE: JULY 2008	PLATE 3



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

DATA AND METHODOLOGY USED TO DEVELOP THIS MAP ARE PRESENTED IN THE FOLLOWING:

Seismic Hazard Evaluation of the City and County of San Francisco, California: California Division of Mines and Geology, Open-File Report 2000-009.

0 1 mile

0 1km

SOURCE: California Division of Mines and Geology, 2000, City and County of San Francisco Seismic Hazards Zones, Released November 17, 2000.

SAN FRANCISCO SEISMIC HAZARD MAP VAN NESS AVENUE BRT EIR/EIS		AGS, Inc. CONSULTING ENGINEERS
JOB NO. KG0205	DATE: JULY 2008	PLATE 4