Performance of Improved Ground During the 2001 Nisqually Earthquake

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ABSTRACT

Several sites in the Seattle area of Washington incorporated ground improvement as liquefaction mitigation or to increase bearing capacity prior to the 2001 Nisqually earthquake. Facilities with improved ground include an earthen dam, a waste repository embankment, lightweight and large plan structures, and bridge columns and approaches. The sites were improved using vibro-replacement stone columns, vibroflotation, or deep dynamic compaction. All sites performed extremely well, despite evidence of liquefaction and minor structural damage nearby. In this paper, 10 sites are summarized, and the performance of three sites located near liquefied or damaged areas will be described in detail. The detailed sites include a large plan commercial property on liquefiable fill improved to a limited lateral extent, a lightweight tilt-up structure located near evidence of liquefaction at King County International Airport, and an earthen dam with its toe retrofitted using vibro-replacement stone columns.

INTRODUCTION

The M_w=6.8 February 28, 2001 Nisqually, Washington earthquake provided yet another set of examples of the successful use of ground improvement to mitigate the detrimental effects of strong ground shaking on structures of many types. Several sites in the Seattle area of Washington State incorporated ground improvement as liquefaction mitigation or to increase bearing capacity prior to the earthquake. Facilities with improved ground tested by the earthquake include an earthen dam, a waste repository embankment, lightweight and large plan structures, and bridge columns and approaches. The sites were improved using vibro-replacement stone columns, vibroflotation, or deep dynamic compaction.

Tables 1 and 2 contain site and performance details for 10 different locations. Table 1 lists the site name, location, facility type, improvement method, and soil characteristics and properties before and after the earthquake. Table 2 consists primarily of earthquake and performance data, including peak acceleration from the nearest strong motion recording station, and performance evidence within and nearby the improved zone. References are listed in Table 2.

The performance of three sites located near liquefied or damaged areas are described in detail in the following sections. The detailed sites include a large plan commercial property on liquefiable fill improved to a limited lateral extent (Home Depot), a lightweight tilt-up structure located near the liquefied areas of King County International Airport (Site A), and an earthen dam with its toe retrofitted using vibro-replacement stone columns (Lake Chaplain South Dam).

HOME DEPOT

The Home Depot store is located at the intersection of S. Utah St. and South Lander St. in the SODO area of Seattle, Washington. The site is on liquefiable fill improved with vibro-replacement stone columns in 1992.

Initial Conditions and Liquefaction Potential

The subsurface at the site consists of 1.2 to 1.5 m (4 to 5 ft) of medium dense to dense granular fill (SM) with some brick and construction debris underlain by approximately 6 m (20 ft) of loose to medium dense sand (SP) or silty sand (SP-SM). Dense to very dense sand is encountered at a depth of approximately 7.6 m (25 ft). Most of the grain size analyses performed on material from the fill layers showed from 3 to 14 percent fines (passing the No. 200 sieve).

Figure 1 shows Dutch Cone Penetration Test results before and after ground improvement. Pre-improvement tip resistances range between 2870 and 4790 kPa (30 and 50 tsf) in the upper 7 m (23 ft). SPT N values in the loose to medium dense sands ranged from 5 to 15 blows per foot. A 30 to 60 cm (1 to 2 ft) thick layer of silty sand (24% passing the No. 200 sieve) was encountered at a depth of approximately 4.3 m (14 ft) in this profile. During drilling, existing grade was at elevation 1.2 m (4 ft) and groundwater was located at approximately 1.5 m (5 ft) below ground surface.

The geotechnical engineer determined that the zone of loose to medium dense sands between 1.5 m and 7.6 m (5 and 25 ft) would be susceptible to liquefaction under the design earthquake.

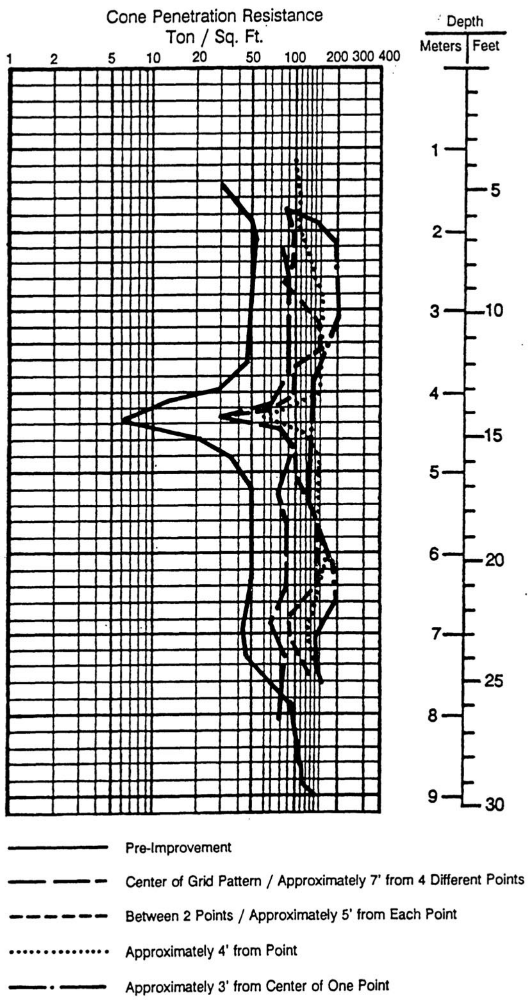

Fig. 1. Dutch cone penetration test results, before and after ground improvement (Campbell and Koelling, 1993)

Superstructure and Foundation

The site includes a single story retail structure, 9290 m^2 (100,000 sq ft) on shallow footings with slab-on-grade construction with a 2787 m^2 (30,000 sq ft) garden center. The building was designed with wall loads of 6.1 N/m (4.5 klf), column loads of 534 kN (120 kips), and slab loading of 31 kPa (650 psf). The allowable design loading after improvement was 144 kPa (3000 psf).

Ground Improvement Goals, Methods, and Construction Procedures

Ground improvement was performed at the site to reduce static settlement and to mitigate the effects of liquefaction. Ground improvement using vibro-replacement stone columns and spread footings with slab-on-grade construction was the chosen alternative because of the short construction time and low cost. Surcharging and spread footings with slabs-on-grade, surcharging and piles with slabs-on-grade, and full pile support with floor slabs were also considered. Deep dynamic compaction was evaluated as an improvement method but deemed unsuitable because of the potential adverse effects of vibration on nearby sites.

The minimum densification criterion set by the geotechnical engineer was an SPT N value of 25 blows per foot or an equivalent CPT q_c of 8425 kPa (88 tsf) down to a depth of 7.6 m (25 ft). If the densification criterion was met, the estimated total settlement would be less than one inch, and differential settlements less than one-half inch after improvement to the recommended criteria.

Vibro-replacement stone columns were installed in a square pattern with 3 m (10 ft) grid spacing. After grading the site with 60 cm (2 ft) of loose granular fill, the improved depth was 8.1 m (26.5 ft). The bottom feed method was used in the lower 4.6 m (15 ft), switching to top feed in the upper 3 m (10 ft). The stone column diameter was about 90 cm. The improved zone included a 3 m (10 ft) wide area beyond the building footprint on three sides. On the east side along the Garden Center, the improved zone stopped short of the building footprint because of the presence of utilities along S. Utah St, as shown in Fig. 2.

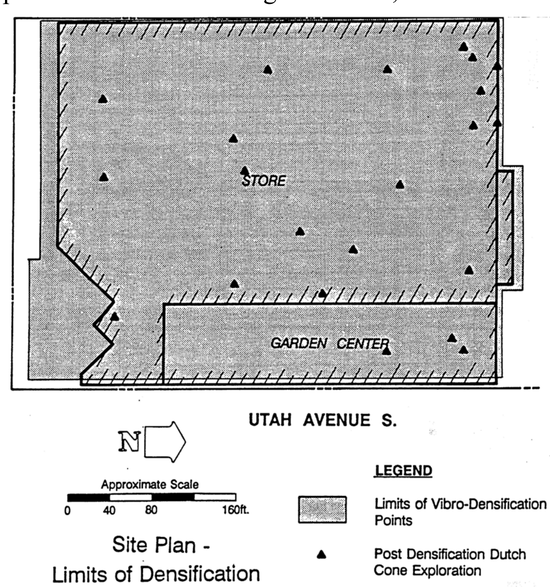

Fig. 2. Site plan and limits of densification (Campbell and Koelling, 1993)

Results from Dutch cone penetration tests (CPTs) before and after improvement are shown in Fig. 1. After improvement, q_c

was as high as 19,150 kPa (200 tsf; N > 55 blows/ft) at the center of the column, between 14,360 and 19,150 kPa (150 to 200 tsf; N = 40 – 55 blows/ft) within one column radius of the stone column, from 9,570 and 14,360 (100 to 150 tsf; N = 28 to 40 blows/ft) within one column diameter, and from 7,660 and 9,570 kPa (80 to 100 tsf; N = 23 – 28 blows/ft) diagonally in between the stone columns. Thus the improvement criterion was satisfied at all locations and depths, except for the 30 to 60 cm silty sand seam at 4.3 m (14 ft) below ground surface.

Performance During the 2001 Nisqually Earthquake

The site is located approximately 76 kilometers north of the epicenter of the earthquake. A recording station at Kimball Elementary in Seattle (N47.57526°, W122.3017°) on the University of Washington Pacific Northwest Seismograph Network measured a peak ground acceleration of 0.092g NS, 0.135g EW, and 0.047g UD. For the east-west component, the Bolt-bracketed duration of this recording was 0.2 seconds and the Arias Intensity was 21.6 cm/s (Rodriguez-Marek, 2001). The recording station is situated on Quarternary till, has a reported hypocentral distance of 77.4 km, and is located about 3.5 km northeast of the site.

The Home Depot store was not damaged, and was open for business shortly after the earthquake. No cracks in the floor slabs were observed from inside the store. No settlement or ground surface disruption was observed or reported within the improved area.

A 10 to 15mm wide crack (Fig. 3) in the sidewalk propagating north-south along the east side of the garden center was caused by the earthquake. The crack was approximately 19 m long and located outside the improved zone in the area where the extent of ground improvement was limited by the presence of utilities. Tension cracks and a large hump ran north-south along S. Utah St. Another 10 to 20 mm wide crack appeared on the sidewalk along the east side of the street, along the utility pole line, running for a distance of about 50 m.

Employees of nearby businesses reported cracks in floor slabs, differential settlements, and temporary uplift of floor slabs during the earthquake. A water main break was reported nearby and a worker observed a 4 ft high geyser during the earthquake that may have been related to the water main rupture. Several buildings in the vicinity of the Home Depot suffered various degrees of damage, including cracking and collapse of nonstructural, unreinforced masonry.

SITE A, KING COUNTY AIRPORT

Site A is located adjacent to King County International Airport (Boeing Field) in Seattle. The facility was under construction at the time of the earthquake. The ground supporting the moment resisting frame structure was improved with vibro-replacement stone columns.

Fig. 3. Crack in sidewalk along east side of Garden Center, looking north, outside improved zone

Initial Conditions and Liquefaction Potential

According to subsurface explorations performed by GeoEngineers in 1999, the subsurface conditions are relatively uniform across the site and consist of soft silt and loose silty fine sand fill to a depth of about 1.5 m (4 to 5 ft) below ground surface, underlain by alluvial soil from 1.5 to approximately 24 m (5 to 80 ft), consisting of an upper coarse-grained unit and a lower fine-grained unit. The upper unit, 1.5 to 14.8 m (5 to 48.5 feet) below ground surface, consists of interbedded layers of sand, sand with silt, and silty sand. The sand layers are typically very loose to medium dense with variable sand and gravel content. The silt layers are typically soft with variable sand and gravel content. Shell fragments and organic matter were found in sand layers at 8.4 to 9.6 m (27.5 to 31.5 ft) depth. A sample obtained at 3 m depth consisted of black fine to medium sand (SP) with D_{90} = 1.1 mm, D_{50} = 0.5 mm, D_{10} = 0.24 mm. At 10.7 m (35 ft), the sample was a dark brown-black fine sandy silt (ML) with D_{50} = 0.075 mm. Based on GeoEngineers (1999) boring logs, the corrected SPT $N_{1,60}$ values in this unit range from 3 to 24 with an average of 12 blows per foot.

The lower unit consists of fine-grained alluvium from about 14.8 to 24 m (48.5 to 79 ft) below ground surface, with layers of very soft silt and elastic silt with variable sand content. Occasional fine gravel was encountered from about 23 to 24 m (75 to 79 ft), organic matter at about 18 m (60 ft), and shell fragments at about 21 m (70 ft). Glacially consolidated sediments exist from 24 to 29.7 m (79 to 97.5 ft) and hard siltstone at 30 m (100 ft). Groundwater was encountered at about 2.9 m (9.5 ft), although estimates vary from 2.1 to 3 m (7 to 10 ft) below ground surface. CPT results prior to improvement are shown in Fig. 4.

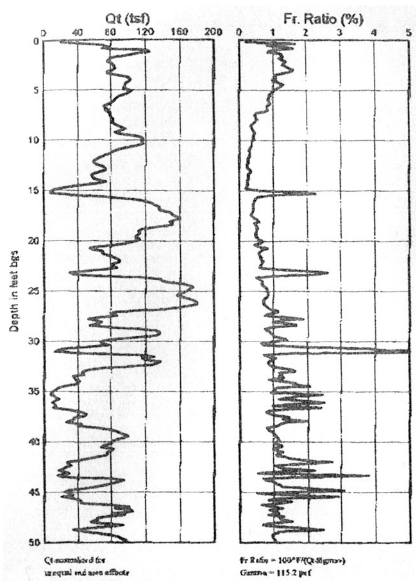

Fig. 4. Cone penetration testing before ground improvement (GeoEngineers, Inc., 2000)

Based on the subsurface exploration, laboratory testing, and a site-specific ground response analysis using acceleration time histories scaled to 0.33g, the geotechnical engineer determined that potentially liquefiable sand extends from about 2.7 m (9 ft) to about 24 m (48.5 ft) below the existing ground surface. Settlement on the order of 20 to 25 cm (8 to 10 inches) and differential settlement of 10 to 12.5 cm (4 to 5 inches) was predicted.

Superstructure and Foundation

The building is 72 m (236 ft 5 inches) long by approximately 43 m (140 ft) wide on the north side and 47 m (155 ft) wide on south side, with a staging area on the west side. The structure is a moment resisting frame.

The exterior walls are supported by a 60 cm by 1 m (2 ft by 3.5 ft) strip footing. An interior wall along the front of the building is supported by 2m by 2m by 60 cm (6.5 ft by 6.5 ft by 2 ft) spread footings connected with a grade beam. The floor is slab-on-grade construction. The spread footings bear on a zone of crushed rock at least 60 cm (2 ft) deep and extends at least 15 cm (6 inches) horizontally beyond the edges of the footings. The allowable bearing pressure for footings supported in this manner was 120 kPa (2,500 psf). This allowable soil bearing pressure applies to the total of dead plus long-term live loads.

Ground Improvement Goals, Methods, and Construction Procedures

The site was improved with 76 cm (30 inch) diameter stone columns. One stone column was installed under each of the load bearing column footings at the front of the building. In addition, stone columns were installed along the perimeter of the building (along the grade beam) and under the hangar door at 2.4 m (8 ft) spacing. The stone columns were not intended to be load bearing. The stone columns were used to improve the soil and reduce the potential for liquefaction-induced settlement. Most stone columns extend to a depth of 12 m (40 ft), however some were obstructed and terminated at shallower depths. All stone columns extended to at least 6 m (20 ft). Post-improvement cone penetration results are shown in Fig. 5.

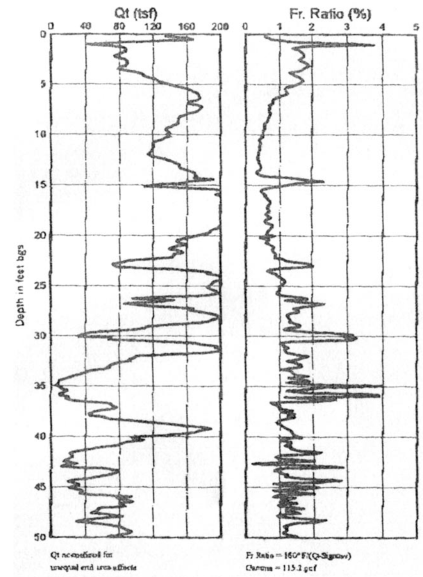

Fig. 5. Cone penetration testing after ground improvement (GeoEngineers, Inc., 2000)

Performance During the 2001 Nisqually Earthquake

The site is located approximately 73 kilometers north of the epicenter of the earthquake. A recording station at King County (N47.5369°, W122.3004°) on the University of Washington Pacific Northwest Seismograph Network measured a peak ground acceleration of 0.17g NS, 0.273g EW, and 0.078g UD. For the east-west component, the relative duration of this recording was 14.5 seconds and the Arias Intensity was 76.6 cm/s, and for the north-south component, the relative duration was 20.1 seconds and the Arias Intensity was 47.9 cm/s (Rodriguez-Marek, 2001). The recording station is situated on Quarternary alluvium, has a reported hypocentral distance of 74.5 km, and is located approximately 1 km east of the site.

No evidence of sand boils, differential settlements, cracking or lateral spreading were observed at the time of the site visit. The geotechnical engineer inspected the site and found no evidence of ground surface disruption. A 1m deep excavation at the rear of the building showed no signs of liquefaction or seeping water. The structural engineer inspected every weld and connection in the new hangar and found no evidence of damage.

Evidence of liquefaction in the form of sand ejecta was present in the nearby runways and grassy areas of Boeing Field (Fig. 6). In an adjacent structure founded on unimproved ground, hairline cracks were found in the tilt-up panel walls.

Fig. 6. Liquefaction ejecta in runway and sink holes in adjacent field, King County International Airport

LAKE CHAPLAIN SOUTH DAM

Lake Chaplain South Dam is located in the Snoqualmie National Forest near Sultan, Washington. The 12.2 m (40 ft) earthen dam was constructed in 1929 and raised in 1945. The toe of the dam was improved using vibro-replacement stone columns.

Initial Conditions and Liquefaction Potential

The subsurface profile consists of loose gravelly and silty sand fill to a depth of 3.7 m (12 ft), underlain by medium dense gravelly and silty sand to a depth of 6.1 m (20 ft), underlain by 9.1 to 12.2 m (30 to 40 ft) of liquefiable loose to medium dense gravelly sand with silt (Bakke et al., undated). Below approximately 20 m (65 ft), the site is underlain by approximately 9 m (30 ft) of dense sand with silt and 6.1 m (20 ft) of very stiff gray clay underlain by dense sand. The groundwater table was found at 6.1 m (20 ft) below ground surface at the time of the subsurface exploration.

The upper 4.6 to 10 m (15 to 33 ft) of soil has fines contents ranging from 19.7 to 31.5 percent and clay fraction (percent smaller than 5 microns) of 6 to 7 percent. The loose soil below the water table has uncorrected SPT N values between 5 and 12 blows per foot. The subsurface soils to the west of the inlet structure were found to be looser than those around the inlet structure and further east. Shear wave velocity testing reported by Bakke et al. (undated) revealed low velocity layers of 202 m/s at 30 cm to 4 m (1 to 13 ft) below ground surface and 228 m/s at 6.1 to 9.4 m (20 to 31 ft) below ground surface.

As a result of a probabilistic seismic hazard assessment and a seismic deformation analysis using two-dimensional finite difference modeling with the program FLAC, the maximum loss of freeboard for the maximum credible earthquake (PGA approximately 0.45g) was predicted to be 2.6 m (8.5 ft) (Bakke et al. undated).

Superstructure and Foundation

The dam is 12.2 m (40 ft) high with a 274 m (900 ft) crest length and 18 to 24 m (60 to 80 ft) crest width. The dam has a toe drain running along its length. The dam impounds 19 billion liters (5 billion gallons) of water and has approximately 2.3 m (7.5 ft) of freeboard. The inlet tower was constructed in 1965.

Fig. 7. Lake Chaplain South Dam, toe area to the west (left) of the inlet tower improved with vibro-replacement stone columns

Ground Improvement Goals, Methods, and Construction Procedures

Because of the potential for slumping due to liquefaction in the soils below the dam and the resultant loss of freeboard that could be caused by an earthquake, the toe of the dam was improved with vibro-replacement stone columns. The improved area begins to the left of the inlet tower (see Fig. 7) and extends to the west (left). The improved zone is approximately 52 m (170 ft) long by 15 m (50 ft) wide. The width of improvement was limited by the presence of the toe drain for the dam on the north side and a 1.8 m (72 inch) diameter concrete pipeline along the south side.

A 12 column test section was used to determine the diameter, spacing and depth of the stone columns. Becker Penetration Tests (BPTs) were performed before and after the test section. BPTs indicated that the soils between 4.6 and 10 m (15 and 33 ft) below ground surface were only moderately densified by the installation, while the soils from 10 to 18.3 m (33 to 60 ft) were significantly denser. Soil sampling and grain size distributions indicated that the upper soil strata contained a higher fines content and clay fraction than previously expected. The fines contents ranged from 19.7 to 31.5 percent and the clay fraction was 6 to 7 percent.

Based on the outcome of the test section and BPTs, bottom feed vibro stone columns were installed in a triangular pattern with 2.1 m (7 ft) spacing on centers to a depth of 19.2 to 19.8 m (63 to 65 ft). The average effective diameter of the stone columns was approximately 1 m (39 to 40 inches) for the interval below 10.7 m (35 ft) and approximately 1.2 m (48 inches) for the 4.6 to 10.7 m (15 to 35 ft) depth interval.

With ground improvement, the maximum loss of freeboard predicted using modeling was 1.7 m (5.5 ft), which is within the acceptable range of 2.3m (7.5 ft) and adequate to prevent breach of the embankment during the MCE.

Performance During the 2001 Nisqually Earthquake

The site is located approximately 152 kilometers north of the epicenter of the earthquake. A recording station at Monroe Substation (N47.8985°, W121.8889°) on the University of Washington Pacific Northwest Seismograph Network measured peak ground acceleration of 0.155g NS, 0.12g EW, and 0.05g UD. For the north-south component, the Arias Intensity was 19.4 cm/s (Rodriguez-Marek, 2001). This recording station is situated on silt, has a reported hypocentral distance of 116 km, and is located about 9 km south of the site.

No evidence of ground surface disruption, ground cracking, or sand boils was observed or reported within or near the dam.

Cracks appeared in the unreinforced masonry on the inlet structure as a result of the earthquake. Demolition of the inlet structure was being planned prior to the earthquake.

SUMMARY

Without exception, sites with ground improvement to prevent liquefaction and minimize settlements performed very well when tested by the February 28, 2001 $M_w = 6.8$ Nisqually, Washington earthquake. No settlement, ground surface disruption, or damage to structures was observed within any of the improved zones. At some sites, evidence of soil strength loss or liquefaction in the form of ground cracking, sink holes, and sand ejecta was found adjacent to or in the vicinity of the improved zones.

ACKNOWLEDGMENTS

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Table 1. Ground Improvement Site Data

Site No	Site Name	Lat, Long, City	Improvement Method	Project	Foundation	Soil Description and Pre-Improvement Properties	Post-Improvement Properties
1	ASARCO Tacoma Smelter OCF	N47.29937 W122.51022 Ruston, WA	Deep Dynamic Compaction under berm footprint	Containment facility embankment	n/a (earthen berm)	4m sand and gravel fill over 1-3m marine clayey silt underlain by glacial deposits, $N_{1,60cs}$ =9-121 bpf in upper 5m, GWT 3m bgs	Upper 2m very dense, compaction not as effective in silt layer, increase in $N_{1,60cs}$ =1-13 bpf in upper 5m
2	Ash Grove Cement Co. Storage Dome	N47.56950 W122.34047 Seattle, WA	VR Stone Columns to 7m depth, 3m beyond ring	Ring foundation storage dome	Shallow ring	2-3m silty sand fill over 1m soft sandy silt over up to 12m loose to med dense dark gray fine sand, N=8-17 bpf, GWT 2-3m bgs	No quantitative data available; improvement was effective at densifying loose sand
3	AT&T Wireless Services Tower	N47.19768 W122.21335 Sumner, WA	VR Stone Columns to 10m depth and 5m outside mat	Transmission tower base	Shallow mat	60cm fill over 8m very loose fine to med dense alluvial sand over med dense silty gravelly sand to 15m, N=1-10 bpf, GWT 3.7m bgs	N = 4-28 bpf, average increase of 5 bpf
4	1st Avenue Bridge	N47.54 W122.34 Seattle, WA	Gravel drains (VR Stone Columns) to 12.2m depth	Structural earth wall for bridge	MSE wall mat	3m clayey silt fill over 1.5m silty fine sand fill over 5.8m loose to med dense clean to slightly silty fine sand grading to dense at depth, N=2-17 bpf	No data available
5	Home Depot	N47.57952 W122.33575 Seattle, WA	VR Stone Columns to 8m, lateral extent limited	Large plan commercial building	Shallow footings	1.5m med dense granular fill over 6-7m loose to med dense sand over dense sand, N=5-15 bpf, Dutch cone q_c=30-50 tsf, GWT 1.5m bgs	N=23-28 bpf, Dutch Cone q_c=80-100 tsf in between columns
6	Klickitat Avenue Overcrossing	N47.57623 W122.35624 Seattle, WA	VR Stone Columns to 12.2m depth under wall footprint	MSE wall overcrossing approach	MSE wall mat	3-5m loose to med dense clean to silty sand hydraulic fill over at least 34m alluvial sand grading from loose to dense, GWT 1.8-3.4m bgs	No data available
7	Lake Chaplain South Dam	N47.94452 W121.82931 Sultan, WA	VR Stone Columns to 18m, 15m by 52m area at toe of dam	Earthen dam toe	n/a (earthen dam)	Silty sand fill to 3.7m over 12 to 15m loose to med dense gravelly silty sand, V_s=202-228 m/s, N=5-12 bpf, GWT 6m bgs	BPTs performed but data not provided; adequate densification achieved
8	Novelty Bridge	N47.70918 W121.99651 Duvall, WA	VR Stone Columns outside sheet pile wall to 4m depth	Bridge abutment	Steel pipe piles enclosed in sheet pile wall	2m med stiff to soft sandy silt over 5.5m loose to dense sand with silt over med dense silty sand with silt interbeds, N=1-9 bpf	N=8-23 bpf
9	Pier 86 Grain Terminal	N47.63683 W122.37202 Seattle, WA	Vibroflotation to 8.5m depth, lateral extent unspecified	Grain silos	Shallow	Loose sand to 8.5m	Relative density of 85% and bearing capacity of 383 kPa (8,000 psf)
10	Site A	Seattle, WA	VR Stone Columns to 12m depth under grade beams and footings	2-story light moment resisting frame	Shallow footings and strip	1.5m loose silty fine sand fill over 24m alluvial soil, coarse to about 15 m, $N_{1,60}$=3-24 bpf in coarse unit, GWT 2-3m bgs	CPT q_c increase of 40 to 80 tsf

Table 2. Earthquake Performance Data

Site No	Site Name	Epicentral Distance	Nearest Strong Motion Recording, PGA, and Arias Intensity	Performance of Improved Area	Performance of Nearby Unimproved Areas	References
1	ASARCO Tacoma Smelter OCF	31 km NW	6 km SE; UW Univ. of Puget Sound, Tacoma Station on till (Qvt); 0.06g NS PGA; 46.2 cm/s EW AI	No ground displacement or signs of liquefaction	Loss of riprap and slag into the bay in low tide area 1 km from site	Doughton (1999), Hydrometrics (1997), Hydrometrics (2000), Kennedy/Jenks (undated)
2	Ash Grove Cement Co. Storage Dome	74 km N	4 km NE; UW Kimball Elementary Station on till (Qvt); 0.135g EW PGA; 21.6 cm/s EW AI	No ground displacement or signs of liquefaction; minor cracking in dome	Signs of liquefaction found in reclaimed areas within 3 km of the site	AGI Technologies (1997)
3	AT&T Wireless Services Tower	55 km N	15 km S; UW East Sheriff Precinct, Puyallup Station on soil (Qvr); 0.21g NS PGA; 50.75 cm/s EW AI	No ground displacement or signs of liquefaction	No ground displacement or signs of liquefaction	AGRA Earth & Environmental, Inc. (1998)
4	1st Avenue Bridge	71 km N	7 km N; UW Kimball Elementary Station on till (Qvt); 0.135g EW PGA; 21.6 cm/s EW AI	No ground displacement or signs of liquefaction	No ground displacement or signs of liquefaction	Shannon & Wilson (1994)
5	Home Depot	76 km N	3.5 km NE; UW Univ. of Puget Sound, Tacoma Station on till (Qvt); 0.06g NS PGA; 46.2 cm/s EW AI	No ground displacement or signs of liquefaction, no structural damage	Ground cracks near edge of improved area, evidence of liquefaction within 1 km of site, structural damage to brick masonry buildings nearby	Campbell and Koelling (1993), Scott (1992), Hayward Baker (undated)
6	Klickitat Avenue Overcrossing	74 km N	3 km W; USGS West Seattle Fire Station on till (Qvt); 0.146g PGA mean	No ground displacement or signs of liquefaction, no damage to wall	Evidence of liquefaction within 2 km of the site	Shannon & Wilson (1997)
7	Lake Chaplain South Dam	152 km N	9 km S; UW Monroe Substation on silt; 0.155g NS PGA; 19.4 cm/s NS AI	No ground displacement or signs of liquefaction, no increased turbidity	Cracks in brick masonry of inlet structure	Bakke et al. (undated)
8	Novelty Bridge	114 km N	30 km S; UW Monroe Substation on silt; 0.155g NS PGA; 19.4 cm/s NS AI	No ground displacement or signs of liquefaction	No ground displacement or signs of liquefaction	HWA Geosciences (2000), King County Dept. of Transportation (1998)
9	Pier 86 Grain Terminal	81 km N	2 km E; UW Queen Anne Station on soil (Qva); 0.114g NS PGA; 31.7 cm/s NS AI	No ground displacement or signs of liquefaction	No ground displacement or signs of liquefaction	Vibroflotation Foundation Co. (1970)
10	Site A	73 km N	1 km E; UW King County Station on soil (Qva); 0.273g EW PGA; 76.6 cm/s EW AI	No ground displacement, signs of liquefaction, or structural damage	Evidence of liquefaction in runways and fields nearby, cracks in adjacent tiltup building	GeoEngineers (1999), GeoEngineers (2000)