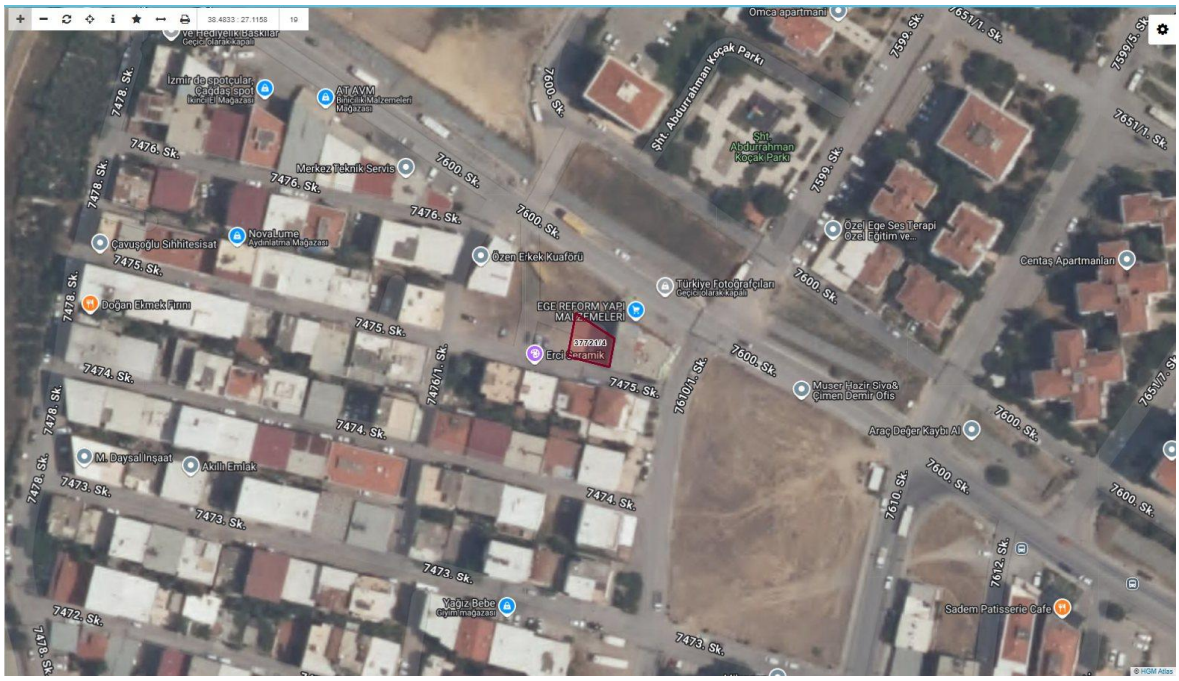
California Province Alameda County District

Book 48, Page 3216 Sheet Block 12 Block Lot 7 (APN 048-3216-012-00) Parcel

Smith Residence – Foundation Study Project

JET GROUTING

GROUND IMPROVEMENT REPORT



Date: 10.11.2026

Medra Mühendislik

DESIGN ENGINEER

Gökhan Demirbaş

TABLE OF CONTENTS

SYMBOLS AND UNITS 5
1. INTRODUCTION 7
2. SITE AND STRUCTURE INFORMATION 7
2.1. Site Information 7
2.2. Structure Information 7
3. SOIL INFORMATION 7
4. SELECTED IMPROVEMENT METHOD AND IMPROVEMENT PARAMETERS 8
4.1. Selected Improvement Method 8
4.2. General Application Details 9
4.3. Selected Improvement Parameters 11
5. GEOTECHNICAL ANALYSES AND CHECKS 13
5.1. Prevention of Liquefaction 13
5.1.1. Liquefaction-Induced Settlement — Before / After Improvement 14
5.2. Bearing Capacity of a Single Jet Grout Column 15
5.3. Total Bearing Capacity and Structural Load Check 16
5.4. Group Effect and Block Failure Check 16
5.5. Column Shear/Tension (Structural) Check 16
6. POST-IMPROVEMENT SOIL PARAMETERS AND STRUCTURAL DESIGN 17
6.1. Strength and Stiffness Properties of the Soil After Improvement 17
6.2. Structural Settlement After Improvement 18
6.3. Bearing Capacity After Improvement 18
6.4. Modulus of Subgrade Reaction After Improvement 18
7. APPLICATION PLAN AND CONSTRUCTION CONTROLS 19
7.1. Application Plan and Details 19
7.1.1. Cement 21
7.1.2. Water to Be Used in the Grout 22
7.2. Construction Controls 22
7.2.1. Trial Columns and Diameter Control 22
7.2.2. Compressive Strength 22
7.2.3. Continuity and Horizontal Deviation of the Columns 22

8. CONCLUSION AND RECOMMENDATIONS 23
9. COST SUMMARY (FOR INFORMATION ONLY) 23
10. REFERENCES 23

LIST OF FIGURES

Figure 1 Application Limits of Ground Improvement Methods 9
Figure 2 Jet Grout Construction Method and Plant Layout 10
Figure 3 Layout Plan Beneath the Foundation 20
Figure 4 Application Section (A) 20
Figure 5 Application Section (B) 21

LIST OF TABLES

Table 1 Symbol Table 5
Table 2 Structure Information 7
Table 3 Idealized Soil Profile and Parameters 7
Table 4 Ranges of Application Parameters of Jet Grouting Methods (Croce et al. 2014) 10
Table 5 Jet Grouting Application Parameters 10
Table 6 Typical Diameter Values for Jet Columns (Croce et al. 2014) 11
Table 7 Dimension and Layout Parameters 11
Table 8 Strength Parameters of Jet Grout Columns for Different Soil Types 12
Table 9 ßE Values Determined in Different Soil Types 12
Table 10 Calculated Area Replacement Ratios for Soil Layers 12
Table 11 Shear Modulus Ratio for SPT Depths 13
Table 12 Shear Stress Reduction Factor for SPT Depths 13
Table 13 Liquefaction Check Before and After Improvement (TBDY 2018) 13
Table 13a Liquefaction-Induced Settlement — Before / After 14
Table 14 Unit Skin Friction (τL) and End-Bearing Resistance (pL) of Jet Columns (Bustamante 2002) 15
Table 15 Single Column Capacity Calculation 15
Table 15a Capacity Summary 15
Table 15b Column Shear/Tension Check 16

Table 16 Unit Weights of the Composite Soil 17
Table 17 Cohesion of the Composite Soil 17
Table 18 Shear Strength Angles of the Composite Soil 18
Table 19 Elastic Modulus of the Composite Soil 18
Table 20 Settlement-Dependent T Factor (Bowles, Foundation Analysis and Design, Eq. 9-9) 19
Table 21 Post-Improvement Design Values 19
Table 22 Cement Type Selection Depending on the Sulfate Concentration in Soil and Water (USBR, 1981) 21
Table M1 Cost Items (excluding VAT) 23

SYMBOLS AND UNITS

The symbols used in this report, their descriptions and units are given below. Subscripts: s = soil, kol = column, komp = composite (improved) soil.

Table 1 Symbol Table

Symbol	Note	Unit
γ (γnatural, γsaturated)	Natural / saturated unit weight of the soil	t/m³
c, c_s'	Zeminin (efektif) kohezyonu	t/m²
c_u	Undrained shear strength	t/m²
ϕ ($Φ_s$)	Internal friction angle of the soil	°
E, E_s	Modulus of elasticity (deformation) of the soil	MPa
ν ($ν_s$)	Poisson's ratio	-
G, G_s	Shear modulus of the soil	MPa
SPT-N, N1.60f	Standard penetration blow count / overburden- and energy-corrected, fines-corrected value	darbe/30 cm
$σ'_v$	Effective vertical stress	t/m²
GWL	Groundwater level (depth)	m
q_t, q_t(iyi)	Allowable bearing capacity — before / after improvement	t/m²
N_s, K_s(iyi)	Modulus of subgrade reaction — $K_s(imp) = T \cdot γ_{Rv} \cdot q_t(imp)$	t/m³
$γ_{Rv}$	Bearing capacity resistance factor (TBDY 2018); $γ_{Rv} \cdot q_t$ is the characteristic bearing capacity	-
T	Settlement-dependent subgrade modulus factor (Bowles)	-
$δ_T$, $δ_F$	Structural settlement — before / after improvement	cm
FS	Factor of safety (bearing capacity / liquefaction)	-
D, B_{kol}	Column diameter	cm
L	Column length	m
a1, a2 (s)	Column spacings (in two directions) / column spacing	cm
s/D	Spacing / diameter ratio	-
A_s	Unit cell area (area tributary to one column)	m²
A_{kol} (A_c)	Column cross-sectional area	m²
a_r	Area replacement ratio — in this report $a_r = ΣAkol / (Akull − ΣAkol)$ (remaining-soil denominator — non-standard)	-
G_{kol}, G_r	Shear modulus of the column / shear modulus ratio $G_r = G_{kol} / G_s$	MPa / -
N_G	Shear stress reduction factor	-
$γ_r$	Shear strain compatibility assumption in the K_G calculation	-

Ω	Amplification factor of the earthquake shear stress attracted by the column	-
τ_{column}, σtension, f_t	Column shear stress / net tensile stress / allowable tensile strength	t/m²
γ_{kol}, c_{kol}, Φ_{kol}, E_{kol} (Ec)	Unit weight, cohesion, friction angle and modulus of elasticity of the column material	t/m³, t/m², °, MPa
γ_{komp}, c_{komp}, Φ_{komp}, E_{komp}	Composite (improved) soil parameters — $X\cdot(1 - a_r) + Xkol\cdot a_r$	t/m³, t/m², °, MPa
q_s, qtip	Unit shaft friction / unit tip resistance	t/m²
Qs, Qtip	Total shaft friction / tip bearing force	t
Aenv, Atip, h	Column perimeter (lateral surface) area / tip area / effective layer thickness	m², m², m
L/B	Column length / diameter ratio	-
η	Group verimi (Converse-Labarre / AASHTO)	-
q_u	(Target) unconfined compressive strength of the column material	MPa
β_E (ßE)	Modulus of elasticity / unconfined compressive strength ratio, $E = \beta_E \cdot q_u$	-
w/c	Grout water / cement ratio	-
CSR	Earthquake-induced cyclic stress ratio	-
CRR	Cyclic resistance ratio of the soil	-
Kσ	Overburden stress correction factor (applied to CRR)	-
MSF	Magnitude scaling factor	-
amax, M_w	Design peak ground acceleration / moment magnitude	g / -
τ_{deprem}, Tr	Earthquake shear stress / liquefaction resistance of the soil (shear stress)	t/m²
ε_v	Post-liquefaction volumetric strain	%
s (liquefaction)	Liquefaction-induced settlement	cm
H	Layer (slice) thickness	m

1. INTRODUCTION

This report contains the analyses and calculations performed for the jet grouting ground improvement design of the Smith Residence – Foundation Study project planned to be built in California province, Alameda County district, Oakland neighborhood, land registry sheet Book 48, Page 3216, block Block 12, parcel Lot 7 (APN 048-3216-012-00). The jet grouting method is based on a high-pressure cement slurry jet cutting the soil in place to form soil-cement (soilcrete) columns. The columns increase bearing capacity, reduce settlements, and lower liquefaction risk.

2. SITE AND STRUCTURE INFORMATION

2.1. Site Information

The site is registered to California province, Alameda County district, Oakland neighborhood, land registry sheet Book 48, Page 3216, block Block 12, parcel Lot 7 (APN 048-3216-012-00). The X and Y coordinates of the approximate midpoint of the parcel have been read as 38.4600 and 27.1074, respectively. The site is relatively Low Slope / %5–10 in slope.

2.2. Structure Information

The structure is planned as RD-1 (Detached Unit Residential); front setback 20 ft, side 5 ft, rear 15 ft; max height 30 ft. The structural system of the structure has been designed as reinforced concrete. Structural information is given in tabular form.

Table 2 Structure Information

Structure Information			
Foundation Short Side (m)	21	**Foundation Type**	SLAB
Foundation Long Side (m)	39	**Total Structure Weight (t)**	21294
Foundation Area (m²)	819	**Foundation Base Depth (m)**	3
Number of Basement Floors	1	**Foundation Base Level (m)**	-3
Total Number of Floors	6	**Foundation Thickness (m)**	0.8

3. SOIL INFORMATION

The borehole, geophysical and laboratory studies performed on site have been examined in the data and geotechnical reports, and in light of this data an idealized soil profile representing the soil mass has been established. The groundwater level was measured at a depth of 6 m below the ground surface.

Table 3 Idealized Soil Profile and Parameters

Layer No	**Soil Type**	**γnatural [t/m³]**	**γsaturated [t/m³]**	**Φs [°]**	**cs' [t/m²]**	**cu [t/m²]**	**Es [MPa]**	**νs**	**Gs [MPa]**

1	Clayey Sand	1.8	1.9	28	1	-	20	0.33	7.55
2	Low Plasticity Clay	1.85	1.93	22.5	1.45	-	25	0.4	8.93
3	Silty Sand	1.84	1.91	30	0.75	-	30	0.3	11.54
4	Non-Plastic Silt	1.78	1.88	30	0.2	-	35	0.4	12.5

Within the scope of the data and geotechnical report, the pre-improvement soil bearing capacity has been calculated as 37.24 t/m², the modulus of subgrade reaction as 2761.1 t/m³, and the average settlement expected in the structure as 2.9 cm.

4. SELECTED IMPROVEMENT METHOD AND IMPROVEMENT PARAMETERS

4.1. Selected Improvement Method

JET GROUTING has been selected as the ground improvement method; the application system is planned as JET-1 (single fluid). In the application, a high-pressure grout jet is sprayed from nozzles at the tip of the rod lowered into the ground; a cylindrical column is formed by rotating and withdrawing the rod at a controlled rate.

Considering the structural design, foundation type, soil layers and groundwater conditions, ground improvement by the jet-grout method was found appropriate for the site. As shown in the figure below, the jet-grout method can be applied in all soil types except in some special cases (soils consisting entirely of high-plasticity fines).

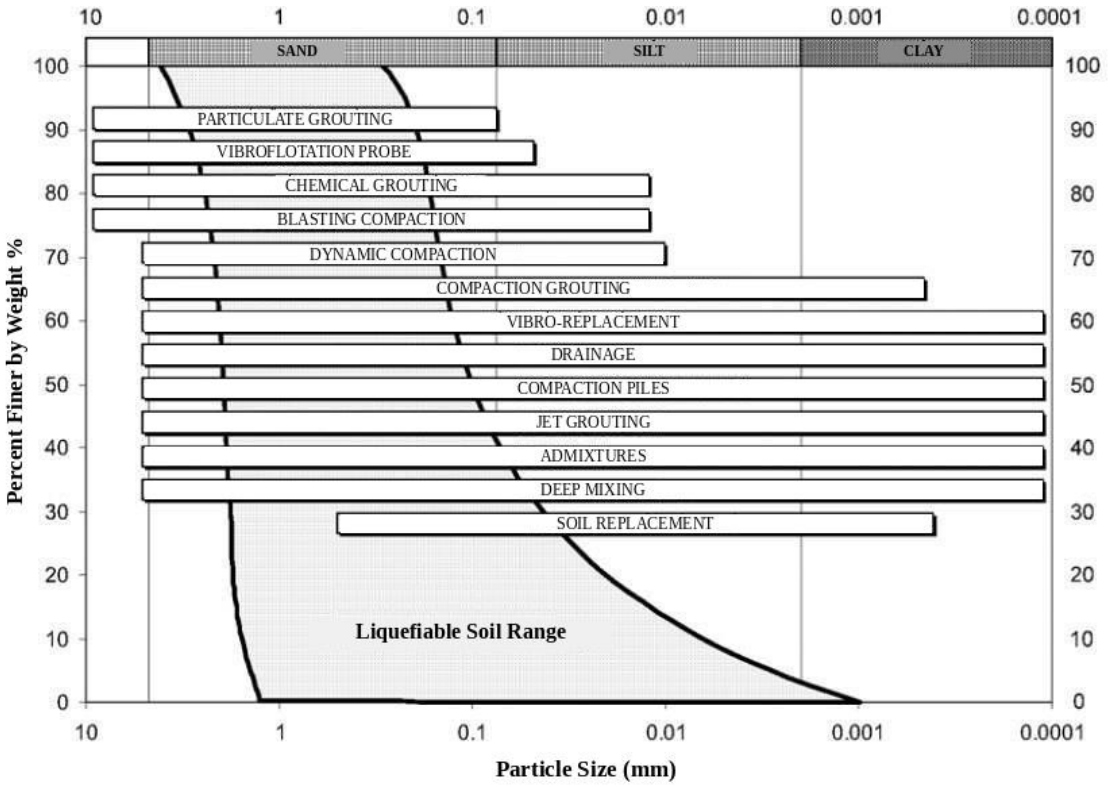


Figure 1 Application Limits of Ground Improvement Methods

4.2. General Application Details

In ground improvement with jet-grout columns, a water–cement mix is injected into the soil at high pressure, and the injected fluid mixes with the soil to form improved zones of approximately circular cross-section. The injection is carried out using high-pressure pumps and special rods. The injection rods are first lowered to the bottom level of the jet-grout column by applying hydraulic water pressure. Injection then starts under high pressure. The rod is rotated at a given speed while being withdrawn towards the ground surface at a prescribed rate, forming the jet-grout columns. In this way cohesion and high compressive strength are imparted to the soil. Although jet-grout columns are stiffer than the soil before improvement, they are assumed to show a certain flexibility during an earthquake and to follow the soil deformations. Owing to this property the columns attract part of the earthquake energy and reduce the cyclic stress ratio that liquefies or softens the soil.

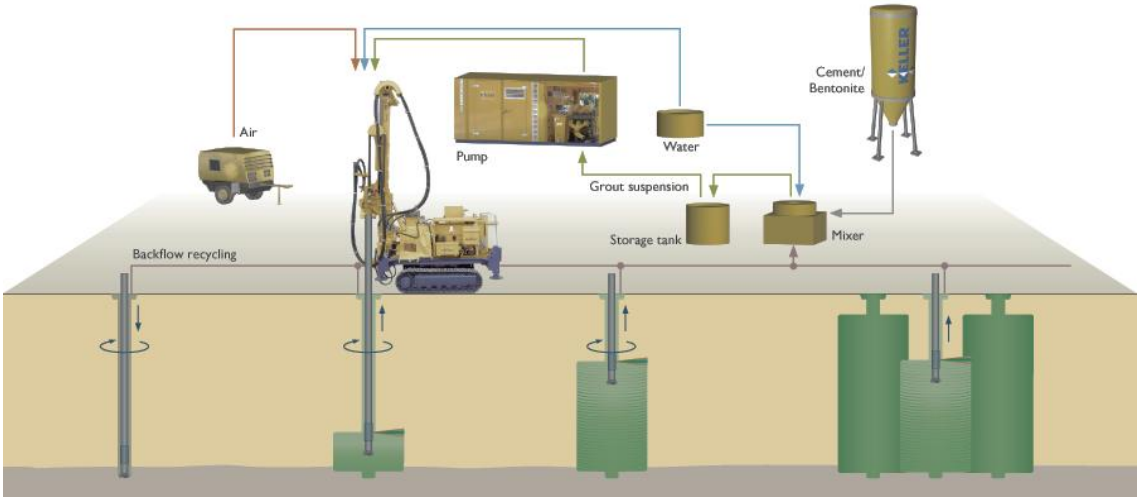


Figure 2 Jet Grout Construction Method and Plant Layout

Jet grouting is carried out by three different methods: JET-1, JET-2 and JET-3. In the JET-1 method the cement–water grout is injected directly into the soil at high pressure. In the JET-2 method a water jet and grout injection are applied to the soil simultaneously. In the JET-3 system the application is made with simultaneous air, water and grout injection. Reference values for the ranges of application parameters of the different methods are given below.

Table 4 Ranges of Application Parameters of Jet Grouting Methods (Croce et al. 2014)

			System		
Parameter	**Symbol**	**Unit**	**Single-fluid**	**Double-fluid**	**Triple-fluid**
Lift step	Δs	mm	40 – 50	40 – 80	40 – 100
Mean lift speed	v_r	mm/s	4 – 10	1 – 8	0.5 – 5
Rotation speed	ω	rpm	5 – 40	3 – 30	1 – 40
Nozzle diameter	d	mm	2 – 8	2 – 8	2 – 8
Number of nozzles	M	-	1 – 2	1 – 2	1 – 2
Injection pressure	p_g	MN/m²	30 – 55	20 – 40	2 – 10
Air pressure	p_a	MN/m²	-	0.5 – 2.0	0.5 – 2.0
Water pressure	p_w	MN/m²	-	-	20 – 55
Grout flow rate	Q_g	L/s	2 – 10	2 – 10	2 – 5
Air flow rate	Q_a	L/s	-	200 – 300	200 – 300
Water flow rate	Q_w	L/s	-	-	0.5 – 2.5
Water/cement ratio	w/c	-	0.60 – 1.25	0.60 – 1.25	0.40 – 1.00

Source: Croce et al. (2014); the information in the original figure has been re-arranged as a table.

The construction parameters to be used for jet grouting are as follows.

Table 5 Jet Grouting Application Parameters

Application system	JET-1 (single fluid)	
Grouting (slurry) pressure	500	bar

Air pressure	-	bar
Water pressure	-	bar
Rod withdrawal rate	25	cm/min
Rotation speed	15	rev/min
Number of nozzles	2	pcs
Nozzle diameter	2.2	mm
Water/cement ratio (w/c)	1	-
Target unconfined compressive strength (q_u)	3	MPa

4.3. Selected Improvement Parameters

The jet-grout application method was selected as JET-1 (single fluid). Examination of typical diameters for different soils and applications shows that the selected method is appropriate.

Table 6 Typical Diameter Values for Jet Columns (Croce et al. 2014)

	Average column diameter (m)			
Jetting System	**Medium-stiff clay**	**Soft silt and clay**	**Silty sand**	**Sand and/or gravel**
Single-fluid	Not recommended	0.4 – 0.8	0.6 – 1.0	0.6 – 1.2
Double-fluid	0.5 – 1.0	0.6 – 1.3	1.0 – 2.0	1.2 – 2.5
Triple-fluid	0.8 – 1.5	1.0 – 1.8	1.2 – 2.5	1.5 – 3.0

Source: Croce et al. (2014); the information in the original figure has been re-arranged as a table.

The dimensional parameters of the jet-grout columns planned on site are targeted as follows.

Table 7 Dimension and Layout Parameters

Column diameter	70	cm
Column length	17.5	m
Column top elevation	-3	m
Kolon alt kotu	-20.5	m
Layout pattern	Grid (square grid)	
Layout spacings ($a_1 \times a_2$)	245 x 245	cm
Unit cell area	6.003	m^2
Total number of columns	144	pcs

The columns are placed below the foundation placed in a SQUARE GRID layout with spacings of 245 cm in the X direction and 245 cm in the Y direction; the unit cell area per column is 6.003 m^2. The layout plan and typical sections are given in Chapter 7.

It is known that jet-grout columns have different stiffness and strength parameters in different soil types. In the calculations the parameters may be selected according to the layers through which the columns pass and which give the lowest strength and stiffness, or they may be selected separately for each layer. In the light of various studies the strength parameters of jet-grout columns have been established as follows.

Table 8 Strength Parameters of Jet Grout Columns for Different Soil Types

Source	Soil Type	ϕ_{MC} (°)	c_{MC} (MN/m²)
Bzowka (2009)	Sandy	58.2	2.3
Croce and Flora (1998)	Silty Sand	26.1	3.2
Mongiovi et al. (1991)	Gravel	52.0	2.1
Mongiovi et al. (1991)	Gravel	42.0	0.3
Mitchell and Katti (1981)	Clay	39.5	0.58
Yahiro et al. (1982)	Sand and clay	28.5	0.4 – 1.0
Miki (1982)	Various	20 – 30	0.7 – 1.0
Yu (1994)	Clay – silty sand	40.6	1.1
Fang et al. (1994a)	Silty sand	35.0	4.2
Fang et al. (1994b)	Clay – silty sand	40 – 44	4.2
Fang and Chung (1997)	Clay and silty sand	38.6	0.8
Fang et al. (2004)	Silt and sand	38.7	0.7
Nikbakhtan and Osanloo (2009)	Clay and sand	42 – 49	0.4 – 0.8
Nikbakhtan and Osanloo (2009)	Clay and sand	25.0	0.77

Source: Croce et al. (2014) and the studies cited therein; the information in the original figure has been re-arranged as a table. ϕ_{MC}, cMC: Mohr-Coulomb internal friction angle and cohesion.

Likewise, the ßE values used to determine the stiffness parameters of jet-grout columns from the target compressive strength have been established by various researchers as follows.

Table 9 ßE Values Determined in Different Soil Types

Source	Definition of E	Soil Type	β_E
Mongiovi et al. (1991)	Tangent (ε not specified)	Gravel	280 – 1000
Lunardi (1992)	Secant (at 40% q_u)	Gravel and sand	500 – 1200
Nanni et al. (2004)	Tangent (ε not specified)	Gravel and sand	440 – 1000
Croce et al. (1994)	Tangent (ε not specified)	Sandy gravel	210 – 670
Croce and Flora (1998)	Secant (at ε_a = 0.01%)	Silty sand	220 – 700
Nanni et al. (2004)	Tangent (ε not specified)	Silty sand	330 – 830
Fang et al. (2004)	Tangent (at 50% q_u)	Silty sand	300 – 750
Fang et al. (2004)	Tangent (at 50% q_u)	Silty sand, silty clay	100 – 300
Lunardi (1992)	Secant (at 40% q_u)	Silt and clay	200 – 500

Source: Croce et al. (2014) and the studies cited therein; the information in the original figure has been re-arranged as a table. $E = \beta_E \cdot q_u$ (q_u: unconfined compressive strength).

Table 10 Calculated Area Replacement Ratios for Soil Layers

Layer No	Soil Type	Bcol [cm]	a1 [cm]	a2 [cm]	Akull [m²]	ΣAkol [m²]	a_r
1	Low Plasticity Clay	70	245	245	819	55.4177	0.0726
2	Silty Sand	70	245	245	819	55.4177	0.0726
3	Non-Plastic Silt	70	245	245	819	55.4177	0.0726

Area replacement ratio definition: Usable area (Ausable) · $a_r = A_{jet} / (A_{soil} - A_{jet})$ (non-standard) (the areas in the table are the areas of this definition).

5. GEOTECHNICAL ANALYSES AND CHECKS

5.1. Prevention of Liquefaction

Sıvılaşma girdileri (deprem kayma gerilmesi τ_{deprem} ve sıvılaşma direnci τ_r) Borehole-1 · GeoteknikRapor sıvılaşma analizinden alınmıştır (TBDY 2018 Bölüm 16.6; S_{DS} = 1.12, M_w = 7.2, amax ≈ 0.4·S_{DS} = 0.448 g); CSR, CRR, rd, MSF ve Kσ ayrıntısı o raporda verilmiştir. Bu raporda yalnız iyileştirme etkisi (K_G) uygulanmıştır.

Since the rigid columns are stiffer than the surrounding soil, they carry a portion of the seismic shear stress; the share falling on the soil is reduced by the K_G factor. The method used in the K_G calculation and the γ_r assumption are determined in the Calculation Settings; the tables below give the program's decision values.

Table 11 Shear Modulus Ratio for SPT Depths

Depth [m]	SPT No	Gcol [MPa]	G_s [MPa]	G_r
1.5	1	200	7.55	26.5
3.25	2	200	8.93	22.4
5	3	200	8.93	22.4
7	4	200	11.54	17.333
10	5	200	11.54	17.333
13.5	6	200	11.54	17.333
15	7	200	11.54	17.333
17.15	8	200	12.5	16
19.5	9	200	12.5	16

Table 12 Shear Stress Reduction Factor for SPT Depths

Depth [m]	SPT No	a_r	G_r	N_G
1.5	1	0.0726	26.5	1
3.25	2	0.0726	22.4	0.3917
5	3	0.0726	22.4	0.3917
7	4	0.0726	17.333	0.4576
10	5	0.0726	17.333	0.4576
13.5	6	0.0726	17.333	0.4576
15	7	0.0726	17.333	0.4576
17.15	8	0.0726	16	0.4788
19.5	9	0.0726	16	0.4788

Table 13 Liquefaction Check Before and After Improvement (TBDY 2018)

Depth [m]	SPT No	N_G	Tdeprem [t/m²]	Tseismic REDUCED [t/m²]	Tr [t/m²]	FS before	FS after	Liquefaction	After Improvement
1.5	1	1	0.77	0.77	0.94	1.22	1.22	YOK (PI) (karar dışı)	YOK (PI) (karar dışı)
3.25	2	0.3917	1.62	0.64	1.58	0.97	2.49	VAR (S PI) (karar dışı)	YOK (S PI) (karar dışı)
5	3	0.3917	2.45	0.96	2.65	1.08	2.76	VAR (S PI) (karar dışı)	YOK (S PI) (karar dışı)
7	4	0.4576	3.35	1.53	1.98	0.59	1.29	YES	NONE
10	5	0.4576	4.6	2.11	2.72	0.59	1.29	YES	NONE
13.5	6	0.4576	5.87	2.69	3.47	0.59	1.29	YES	NONE
15	7	0.4576	6.35	2.91	7.95	1.25	2.73	NONE	NONE
17.15	8	0.4788	6.96	3.33	10.2	1.47	3.06	NONE	NONE
19.5	9	0.4788	7.54	3.61	11.9	1.58	3.30	NONE	NONE

The liquefaction check was performed at 6 points on a SPT elevation basis; after improvement FS ≥ 1.10 at all of them and liquefaction is MITIGATED. 3 row(s) are excluded from the decision because they lie above the groundwater level or within the excavation (marked "excluded" in the table). K_G method: Baez & Martin (strain-compatible) (a_r = 0.0726, mean K_G = 0.4535; Baez & Martin 0.4535 · Rayamajhi 0.924).

5.1.1. Liquefaction-Induced Settlement — Before / After Improvement

The post-earthquake volumetric settlement of the liquefied layers due to volumetric strain has been calculated using the Ishihara–Yoshimine approach (Idriss & Boulanger 2008 closed form). The same relationship is applied twice: the BEFORE-improvement FS and the AFTER-improvement FS reduced by K_G — the comparison is one-to-one.

Table 13a Liquefaction-Induced Settlement — Before / After

Depth [m]	Layer	N1.60f	H [m]	FS (before)	ε_v before [%]	s before [cm]	FS (after)	ε_v after [%]	s after [cm]
7	Silty Sand	14	2.5	0.591	3.02	7.54	1.291	0.39	0.97
10	Silty Sand	15.8	3.25	0.591	2.77	9	1.292	0.42	1.36
13.5	Silty Sand	16.6	2.5	0.592	2.67	6.67	1.293	0.43	1.07
15	Silty Sand	28.1	0.75	1.251	0.44	0.33	2.735	0	0
17.15	Non-Plastic Silt	29.5	2.25	1.466	0.26	0.59	3.061	0	0
19.5	Non-Plastic Silt	30	1.68	1.578	0.19	0.32	3.297	0	0

While the total liquefaction settlement before improvement is 24.45 cm, it drops to the order of 3.4 cm after improvement (reduction ≈ %86). Since K_G is not applied at levels outside the column length, the settlement contribution DOES NOT CHANGE and appears the same in the table.

5.2. Bearing Capacity of a Single Jet Grout Column

The bearing capacity of jet-grout columns is calculated from the friction resistance of the soil they pass through and the tip resistance of the last layer to which they transfer load. The recommended unit friction and tip resistances for different soil types are given below as a function of the SPT-N value.

Table 14 Unit Skin Friction (τ_L) and End-Bearing Resistance (pL) of Jet Columns (Bustamante 2002)

	Unit skin friction τ_L (MPa)				End-bearing resistance pL (MPa)			
Soil Type	**N = 20**	**N = 40**	**N = 60**	**N = 80**	**N = 20**	**N = 40**	**N = 60**	**N = 80**
Silty soil	0.06	0.12	0.18	0.24	1.0	1.9	2.7	3.6
Clayey soil	0.04	0.08	0.12	0.16	0.5	0.9	1.4	1.8
Gravelly soil	0.11	0.23	0.34	0.45	2.6	4.9	7.3	9.6
Sandy soil	0.09	0.18	0.27	0.36	1.3	2.4	3.6	4.7

Source: Bustamante (2002) SPT-N curves; the values are APPROXIMATE readings from the original chart (linear curves); N: SPT blow count. The program's calculation uses its own Bustamante reading.

Using the determined unit friction and unit tip resistance, together with the lateral surface areas depending on the effective thicknesses of the columns within the soil layers and the column tip area, the capacities were calculated as follows.

Table 15 Single Column Capacity Calculation

Layer No	Soil Type	q_s [t/m²]	h [m]	Diameter [m]	Aperim [m²]	Qs [t]	Atoe [m²]	qtoe [t/m²]	Qtoe [t]
1	Low Plasticity Clay	4.35	3	0.7	6.6	28.7	-	-	-
2	Silty Sand	6.62	9	0.7	19.79	131.09	-	-	-
3	Non-Plastic Silt	8.19	5	0.7	11	90.03	0.3848	130.26	50.13

Table 15a Capacity Summary

Total shaft friction	249.82	ton
Toe bearing	50.13	ton
Single column ultimate capacity	299.95	ton
Structural failure limit	118	ton
Factor of safety (FS)	3	-

Single safe capacity — geotechnical (Q/FS)	99.98	ton
Structural design capacity ($\eta \cdot q_u \cdot A/FSm$)	39.24	ton
Single column safe capacity (governing: STRUCTURAL)	39.24	ton
Length/diameter ratio (L/B)	25	-

5.3. Total Bearing Capacity and Structural Load Check

A total of 144 jet grout columns have been installed beneath the foundation. The total column bearing capacity is 3822 tons, and when compared with the total structure weight of 21294 tons obtained from the superstructure analyses, the capacity has been found to be insufficient to carry the structural weight ALONE (composite bearing must be assumed) (group efficiency η and block failure included; sum of singles 5651 t).

Engineering fill beneath the foundation will not be applied.

5.4. Group Effect and Block Failure Check

The columns were assessed as a GROUP, not by the arithmetic sum of their single capacities. The efficiency is the SMALLER of Converse-Labarre and the AASHTO linear transition (safe side; η = 0.68 for s/D = 3.50, Converse–Labarre governs); block failure was checked by solving the outer envelope of the group as a single large column (soil–soil sliding around its perimeter + tip resistance at its base). Σsingle = 5651 t → efficient group capacity 3822 t; block design capacity 36386 t. DECISION: 3822 t (EFFICIENCY-REDUCED SINGLE (η)).

s/D = 3.50 → η = 0.68 [CL 0.68 (16×9) · AASHTO 0.75 → THE SMALLER, Converse–Labarre governs; η is applied to the design capacity (with FS), FS is not applied again]. Σsingle = 5651 t → efficient 3822 t · block 36386 t (FS = 3) → DECISION 3822 t (EFFICIENCY-REDUCED SINGLE (η)).

5.5. Column Shear/Tension (Structural) Check

The seismic shear stress carried by the column is amplified by the Ω factor ($\tau_{column} = \Omega \cdot \tau_{earthquake}$); a TENSION of equal magnitude arises in pure shear, and the axial stress reduces this tension. The net tension has been compared with the allowable tensile strength of the column material: at any elevation where the ratio exceeds 1, the column CRACKS and the shear stress reduction (SSR) benefit at that elevation is lost.

Table 15b Column Shear/Tension Check

Depth [m]	Layer	$\tau_{earthquake}$ [t/m²]	Ω	τ_{column} [t/m²]	$\sigma_{tension}$ [t/m²]	f_t allow [t/m²]	Ratio	Status
3.25	Low Plasticity Clay	1.62	8.77	14.25	14.25	61.18	0.23	OK

5	Low Plasticity Clay	2.45	8.77	21.51	21.51	61.18	0.35	OK
7	Silty Sand	3.35	7.93	26.58	26.58	61.18	0.43	OK
10	Silty Sand	4.6	7.93	36.5	36.5	61.18	0.60	OK
13.5	Silty Sand	5.87	7.93	46.55	46.55	61.18	0.76	OK
15	Silty Sand	6.35	7.93	50.37	50.37	61.18	0.82	OK
17.15	Non-Plastic Silt	6.96	7.66	53.32	53.32	61.18	0.87	OK
19.5	Non-Plastic Silt	7.54	7.66	57.76	57.76	61.18	0.94	OK

The axial compression contribution is NOT INCLUDED (Calculation Settings): in pure shear the principal tension equals the shear stress, so in the table σtension = τ_{column} (safe side).

At all depths the column stresses are below the allowable values; the shear stress reduction (K_G) benefit is also structurally valid.

6. POST-IMPROVEMENT SOIL PARAMETERS AND STRUCTURAL DESIGN

6.1. Strength and Stiffness Properties of the Soil After Improvement

The improved soil parameters are calculated assuming that the soil and the columns form a composite material. Assuming that the improvement is effective over the whole soil depending on the area replacement ratio (a_r), the composite parameters are obtained from the following relations:

$$\gamma_{komp} = \gamma_s \cdot (1 - a_r) + \gamma_{kol} \cdot a_r \quad \cdot \quad c_{komp} = c_s \cdot (1 - a_r) + c_{kol} \cdot a_r \quad \cdot \quad \Phi_{komp} = \Phi_s \cdot (1 - a_r) + \Phi_{kol} \cdot a_r \quad \cdot \quad E_{komp} = E_s \cdot (1 - a_r) + E_{kol} \cdot a_r$$

Table 16 Unit Weights of the Composite Soil

Layer No	Soil Type	γ_s [t/m³]	γcol [t/m³]	a_r	γcomp [t/m³]
1	Low Plasticity Clay	1.85	2.3	0.0726	1.88
2	Silty Sand	1.84	2.3	0.0726	1.87
3	Non-Plastic Silt	1.78	2.3	0.0726	1.82

Table 17 Cohesion of the Composite Soil

Layer No	Soil Type	c_s [t/m²]	ccol [t/m²]	a_r	ccomp [t/m²]
1	Low Plasticity Clay	1.45	54.96	0.0726	5.33
2	Silty Sand	0.75	54.96	0.0726	4.68
3	Non-Plastic Silt	0.2	54.96	0.0726	4.17

Table 18 Shear Strength Angles of the Composite Soil

Layer	Soil Type	Φ_s [°]	Φcol [°]	a_r	Φcomp [°]
1	Low Plasticity Clay	22.5	50	0.0726	40.6
2	Silty Sand	30	50	0.0726	42.84
3	Non-Plastic Silt	30	50	0.0726	42.84

Table 19 Elastic Modulus of the Composite Soil

Layer No	Soil Type	E_s [MPa]	Ecol [MPa]	a_r	Ecomp [MPa]
1	Low Plasticity Clay	25	500	0.0726	59.47
2	Silty Sand	30	500	0.0726	64.11
3	Non-Plastic Silt	35	500	0.0726	68.75

6.2. Structural Settlement After Improvement

Within the scope of the improvement, the modulus of elasticity of the soil as a composite material increases. In addition, the columns transfer the load to depth; therefore the settlement in the soil will be smaller than that calculated with the composite parameters alone. Neglecting the column effect on load spreading, the settlement after improvement was calculated by reducing the settlement before improvement by the composite modulus ratio: $\delta_F = \delta_T$ / (Ekomp / E_s).

δ_T = 2.9 cm (settlement analyzed before improvement)

δ_F = 1.33 cm.

Settlement approach used in the program: simple ratio — $\delta_F = \delta_T \cdot E_s$ / Ecomp; E_s = 29.58 MPa (entered manually), Ekomp = 64.66 MPa (Ekomp/E_s = 2.186).

CAUTION: the stress carried by the column in the elastic stress distribution (204.01 t/m²) exceeds the column's allowable stress (101.97 t/m²) and the n limit is OFF in the Calculation Settings; settlement and bearing capacity have been calculated with a share the column cannot carry (optimistic).

6.3. Bearing Capacity After Improvement

The bearing capacity of the soil increases owing to the better spreading of the load transferred through the columns within the soil, the composite nature of the soil and the column tip resistances.

q_t = 37.24 t/m² (safe bearing capacity before improvement)

q_t(imp) = 39.54 t/m².

6.4. Modulus of Subgrade Reaction After Improvement

After ground improvement the modulus of subgrade reaction increases depending on the bearing capacity and settlement; it increases linearly with increasing bearing capacity and decreasing settlement. The modulus of subgrade reaction is determined from K_s(imp) = T · γ_{Rv} ·

q_t(imp) (the same relation as in the Geotechnical Report; γ_{Rv} = 1.40 is the bearing capacity resistance factor and $\gamma_{Rv} \cdot q_t$ returns the characteristic bearing capacity). The factor T should not be taken as 40 in every case: the modulus of subgrade reaction is a parameter that changes the load transferred to the structure depending on the soil deformation in the superstructure analysis, and taking it lower than it should be does not keep the structure on the safe side but leads to the shear forces acting on the structure being underestimated. The variation of T with settlement is given in the table below.

Table 20 Settlement-Dependent T Factor (Bowles, Foundation Analysis and Design, Eq. 9-9)

Settlement	T
≥ 2.54 cm	40
2.00 cm	50
1.20 cm	83
0.60 cm	160

The T coefficient has been determined as 122.1 (from immediate settlement 0.895 cm; Bowles, Foundation Analysis and Design, Eq. 9-9) using the settlement correlation; K_s(imp) = T · γ_{Rv} · q_t(imp) = 122.1 · 1.40 · 39.54 = 6761.6 t/m³.

Table 21 Post-Improvement Design Values

Soil Bearing Capacity [t/m²]	Modulus of Subgrade Reaction [t/m³]
39.54	6761.6

7. APPLICATION PLAN AND CONSTRUCTION CONTROLS

7.1. Application Plan and Details

The planned column layout, foundation plan and section are shown below. The columns are placed in a SQUARE GRID layout with spacings of 245 cm in the X direction and 245 cm in the Y direction; the unit cell area per column is 6.003 m².

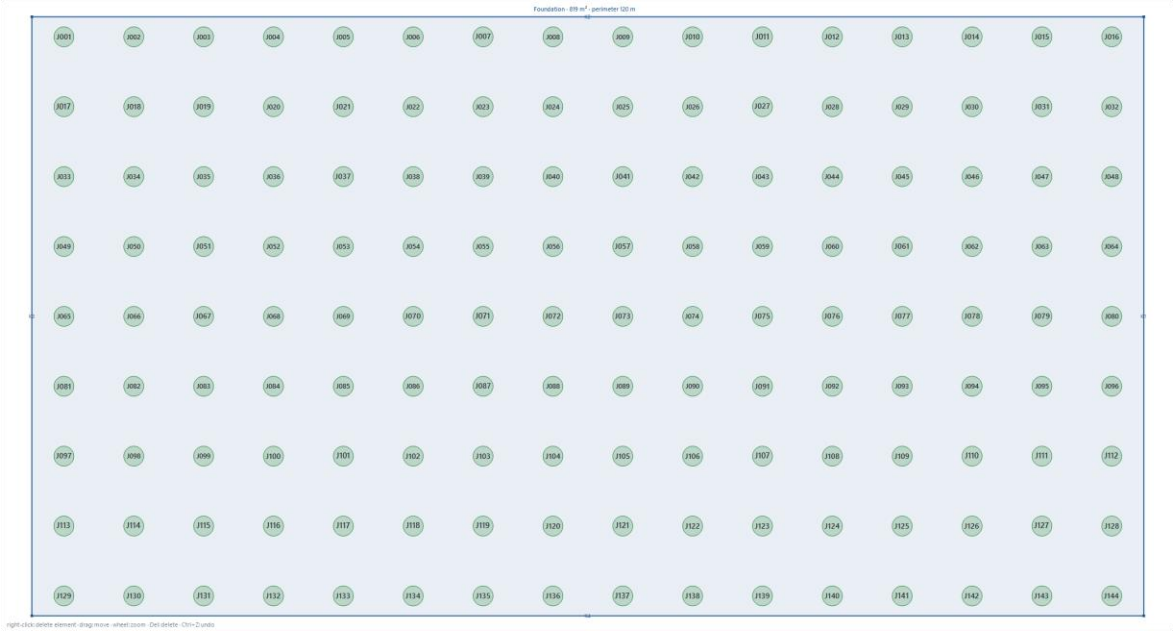
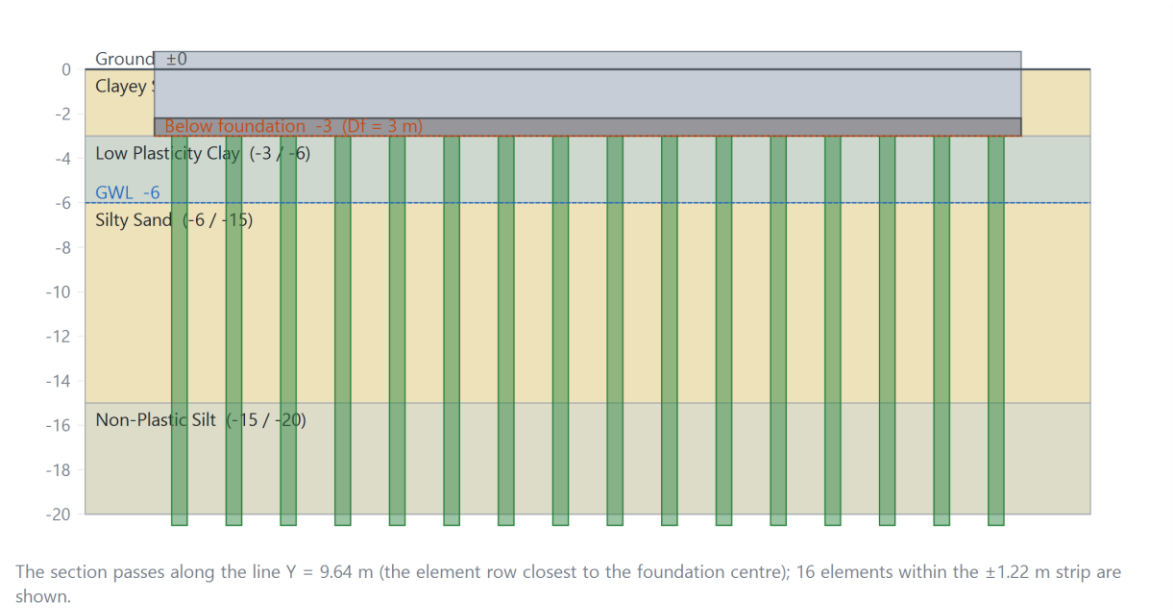
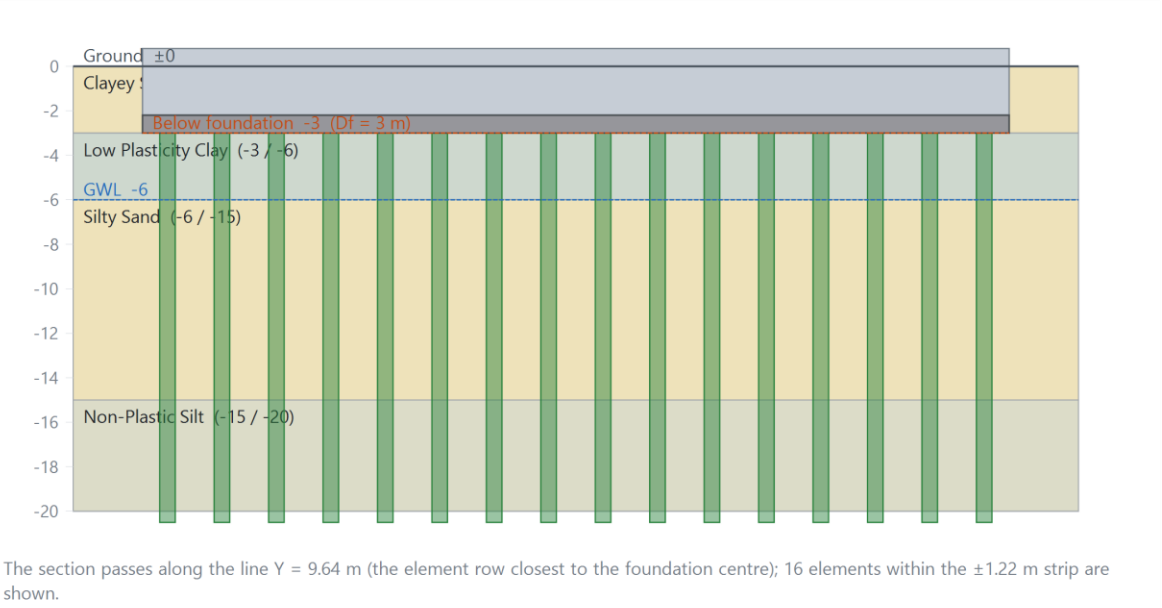


Figure 3 Layout Plan Beneath the Foundation



The section passes along the line Y = 9.64 m (the element row closest to the foundation centre); 16 elements within the ±1.22 m strip are shown.

Figure 4 Application Section (A)



The section passes along the line Y = 9.64 m (the element row closest to the foundation centre); 16 elements within the ±1.22 m strip are shown.

Figure 5 Application Section (B)

Jet grouting is usually carried out before the foundation excavation. The part above the planned top level of the column is drilled without grouting and the application must be stopped at the top level. The columns must be constructed in an alternating sequence; adjacent columns must not be constructed one after another. As the surface is approached, the pressure must be adjusted appropriately to reduce vibration. Since this vibration may have undesirable consequences for adjoining buildings, slope edges and underground and above-ground utilities, it must be carefully controlled by the contractor.

7.1.1. Cement

It is important for construction that the cement is selected considering the presence of groundwater and the sulphate content of the water/soil (usually Portland cement of strength class 42.5 N). The cement type selection depending on the sulphate concentration is given below.

Table 22 Cement Type Selection Depending on the Sulfate Concentration in Soil and Water (USBR, 1981)

Degree of sulfate exposure	Water-soluble sulfate in soil (as SO_4), %	Sulfate in water (as SO_4) (ppm)	Cement type
Negligible	0.00 – 0.10	0 – 150	I
Positive	0.10 – 0.20	150 – 1500	II
Severe	0.20 – 2.00	1500 – 10000	V
Very severe	≥ 2.00	≥ 10000	V + Pozzolan

Source: USBR (1981); the information in the original figure has been re-arranged as a table.

For very severe sulphate contents, the use of pozzolan-blended cement is recommended.

7.1.2. Water to Be Used in the Grout

Care must be taken that the chemical composition of the water used in the grout does not hinder cement hydration and setting. Mains tap and drinking water may be used in the mix. The water must be within the following limits: temperature < 60 °C · sodium/magnesium chloride content < 3% · ammonium sulphate content < 6% · organic matter and clay content < 2 g/L.

7.2. Construction Controls

All construction checks must be carried out by accredited laboratories; all tests must be documented and archived.

7.2.1. Trial Columns and Diameter Control

Trial columns must be constructed taking into account the variations of the soil layers and thicknesses on site; the head zones must be exposed to check whether the target diameters have been reached. The checks must be applied in turn to each trial column after construction so that the parameters are calibrated and defective construction is prevented.

7.2.2. Compressive Strength

After a suitable diameter has been obtained in the trial columns, at least 7 days must be allowed for the columns to reach adequate strength; core samples must then be taken from the columns to check that the target compressive strength has been reached.

7.2.3. Continuity and Horizontal Deviation of the Columns

That the columns do not deviate from the vertical axis and that no discontinuity occurs along their length can be checked by PIT (pile integrity testing) or by continuous coring. Carrying out the tests first on the trial columns and continuing construction in the light of these data minimises errors. In this project PIT shall be performed on 2% of all columns, on at least 3 columns.

Summary checks with the program parameters:

1) At least 3 trial columns shall be constructed on site; the head zones shall be exposed for at least 1.0 m to check the diameter and continuity.

2) From at least 2% of the columns constructed (3 columns), 3 cores per column shall be taken; unconfined compression testing shall be performed on the cores, and it shall be shown that the results satisfy the target q_u = 3 MPa value.

3) The column vertical deviation shall not exceed %1, and the plan position deviation shall not exceed 5 cm.

4) The grout w/c ratio shall be prepared as 1; a density (Baumé) check shall be performed on each shift.

5) PIT (pile integrity test, ASTM D5882) shall be performed on at least 3 columns (2%).

8. CONCLUSION AND RECOMMENDATIONS

In light of the analyses, it has been decided to apply jet grout columns (JET-1 (single fluid)) with a diameter of Ø70 cm and a length of 17.5 m. The columns are placed in a SQUARE GRID layout with spacings of 245 cm in the X direction and 245 cm in the Y direction; the unit cell area per column is 6.003 m²; the total number is 144.

The single column safe bearing capacity has been calculated as 39.24 tons; the post-improvement soil bearing capacity as 39.54 t/m², the expected settlement as 1.33 cm, and the modulus of subgrade reaction as 6761.6 t/m³.

CAUTION — the following checks are NOT SATISFIED; the improvement design is NOT ADEQUATE as it stands and must be reviewed: 1) the stress carried by the column in the stress distribution exceeds the column capacity (n limit off) — the settlement and bearing capacity results are unreliable.

9. COST SUMMARY (FOR INFORMATION ONLY)

The amounts below are APPROXIMATE values calculated using the unit prices defined in the program; they do not include VAT and do not constitute an offer/quotation. This section can be turned off from the section list in the Report window and is CLOSED BY DEFAULT — if amounts are not desired in the report submitted to the authority, it does not appear at all.

Table M1 Cost Items (excluding VAT)

Item	Quantity	Unit	Unit Price [TL]	Amount [TL]
Jet-Grout construction	2520	m	1.500	3.780.000
Empty drilling	432	m	250	108.000
Cement	567.34	ton	4.500	2.553.024
Water	567.34	m³	60	34.040
Trial column	3	pcs	25.000	75.000
Core	9	pcs	3.500	31.500
Mobilization	1	lump sum	350.000	350.000

TOTAL: 6.931.564 TL (excluding VAT) · unit cost 8.463 TL/m².

10. REFERENCES

[1] Croce, P., Flora, A., & Modoni, G. (2014). Jet grouting: Technology, design and control. CRC Press.

[2] Idriss, I. M., & Boulanger, R. W. (2008). Soil liquefaction during earthquakes (MNO-12). Earthquake Engineering Research Institute.

[3] Ishihara, K., & Yoshimine, M. (1992). Evaluation of settlements in sand deposits following liquefaction during earthquakes. Soils and Foundations, 32(1), 173–188.

[4] Ministry of Environment and Urbanisation of the Republic of Turkey. (2018). Turkish Building Earthquake Code (TBDY 2018).