



BAGAHİ ENGINEERING INC.

GEOTECHNICS & FOUNDATIONS

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April 27, 2018

Project No. 212s-200-01

METRO PROPERTIES GROUP, LLC.

4433 Convoy Street, Suite H
San Diego, CA, 92111

Attention: Mr. Kenneth K. Wang

SUBJECT: **UPDATED GEOTECHNICAL REPORT AND INFILTRATION
EVALUATION**
Dolphin Green Car Wash
2190 S. El Camino Real
Oceanside, CA

REFERENCES: See Attached

Dear Mr. Wang:

In accordance with your request and authorization, Bagahi Engineering, Inc., has completed an update of our previous reports for the site (References 2 and 3) and reviewed a recent report by T.I.N. Engineering (Reference 1) related to site permeability. This report summarizes our findings and recommendations. All our previous recommendations remain valid unless as modified herein.

SEISMIC DESIGN PARAMETERS

For seismic design purposes, and utilizing the 2016 CBC edition, the following seismic factors would apply to this site:

Site Class: C

Mapped one Second Spectral Response Acceleration, $S_1 = 0.424$ g

Mapped 0.2 Second Spectral Response Acceleration, $S_s = 1.099$ g

Site Coefficient, $F_a = 1.0$

Site Coefficient, $F_v = 1.376$

5% Design Spectral Response Acceleration, $S_{DS} = 0.732$ g

5% Design Spectral Response Acceleration, $S_{DI} = 0.389$ g

PGA=0.424 g

PGAm=0.424 g

Detail output is presented in Appendix B.

The site is not mapped as prone to liquefaction by State of California Seismic Hazard Zones Map.

SLAB-ON-GRADE

Slab thickness and reinforcement for building slab-on-grade should be designed by the project Structural Engineer in accordance with Section 1805.8.2 of 2016 CBC using an effective plasticity index of 25.

TYPE OF CEMENT

It is recommended that all concrete in contact with subgrade soils meet the minimum requirement of 2016 CBC Section 1904.3 for concrete exposed to at least negligible sulfate exposure.

PERCOLATION

Two field permeability tests were carried out by T.I.N. Engineering Company on near-surface subgrade soils on September 16, 2017. The tests were performed at depths of between 6 to 48 inches. Below 48 inches a bed of tan sandstone and claystone bedrock were encountered. Based on measured permeability rates, a permeability of 0.00418 inch/minute was recommended for hydraulic design in their report of findings. Details of the procedure used and their findings are presented in Appendix C.

The reported measured permeability of 0.00418 inch/minute corresponds to a percolation rate of 0.25 inch/hour. Using a factor of safety of 2 in accordance with Technical Guidance Document worksheet for determining design infiltration rate, the corresponding design infiltration rate is

Mr. Kenneth Wang

Project: Updated Geotechnical Report and Infiltration Evaluation

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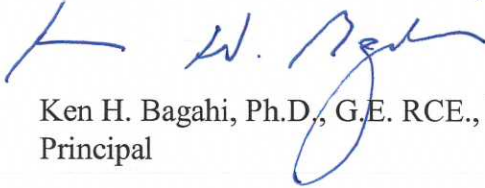
0.125 inch/hour which is considered inadequate for infiltration BMP. Since near surface soils are underlain by a bed of sandstone/claystone bedrock which is expected to have a very low infiltration rate, it is concluded that infiltration BMP is not considered suitable for this site.

Our update and evaluation were performed in accordance with generally accepted practice in the geotechnical field. No warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The opportunity to be of service is appreciated. If you have any questions, please call.

Very truly yours,

BAGAHİ ENGINEERING INC.



Ken H. Bagahi, Ph.D., G.E. RCE., PE.
Principal



KHB/ma

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APPENDIX A

REFERENCES

1. T.I.N. Engineering Company, “ Permeability of On-site Upper Soils for Proposed Car Wash at 2190 S. El Camino Real, Oceanside, CA”, dated September 29, 2017.
2. Bagahi Engineering, Inc., “ Grading Plan Review and Update, Dolphin Green Car Wash, 2190 S. El Camino Real, Oceanside, CA”, dated May 12, 2014.
3. Bagahi Engineering, Inc., “ Updated Geotechnical Report, Dolphin Green Car Wash, 2190 El Camino Real, Oceanside, CA”, dated May 20, 2010.
4. San Marcos Engineering, Inc., “Preliminary Geotechnical Investigation of El Camino Inn Site, El Camino Real/Via Las Rosas, APN 165-012”, dated May 14, 1986.
5. San Marcos Engineering, Inc., “ Grading and Soils Testing Report for Crib Walls of El Camino Real/ Via Las Rosas, Oceanside , CA.”, dated Dec. 14, 1989.
6. San Marcos Engineering, Inc., “After Grading Soils Testing and Monitoring Report for Phase One Grading”, dated Feb. 15, 1989.
- 7.”Grading and Drainage Plans for Dolphin Green Car Wash, 2190 S. El Camino Real, Oceanside, CA”, by Jonathan Sagherian, Sheets 1 through 7, Undated.

APPENDIX B

SEISMIC DESIGN PARAMETERS

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.18731°N, 117.32741°W)

Site Class C – “Very Dense Soil and Soft Rock”, Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_s) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From [Figure 22-1](#) ^[1]

$$S_s = 1.099 \text{ g}$$

From [Figure 22-2](#) ^[2]

$$S_1 = 0.424 \text{ g}$$

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	\bar{V}_s	\bar{N} or \bar{N}_{ch}	\bar{s}_u
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf

Any profile with more than 10 ft of soil having the characteristics:

- Plasticity index $PI > 20$,
- Moisture content $w \geq 40\%$, and
- Undrained shear strength $\bar{s}_u < 500$ psf

F. Soils requiring site response analysis in accordance with Section

See Section 20.3.1

For SI: $1 \text{ ft/s} = 0.3048 \text{ m/s}$ $1 \text{ lb/ft}^2 = 0.0479 \text{ kN/m}^2$

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE _R Spectral Response Acceleration Parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = C and $S_s = 1.099$ g, $F_a = 1.000$

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE _R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and $S_1 = 0.424$ g, $F_v = 1.376$

Equation (11.4-1):

$$S_{MS} = F_a S_S = 1.000 \times 1.099 = 1.099 \text{ g}$$

Equation (11.4-2):

$$S_{M1} = F_v S_1 = 1.376 \times 0.424 = 0.583 \text{ g}$$

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.099 = 0.732 \text{ g}$$

Equation (11.4-4):

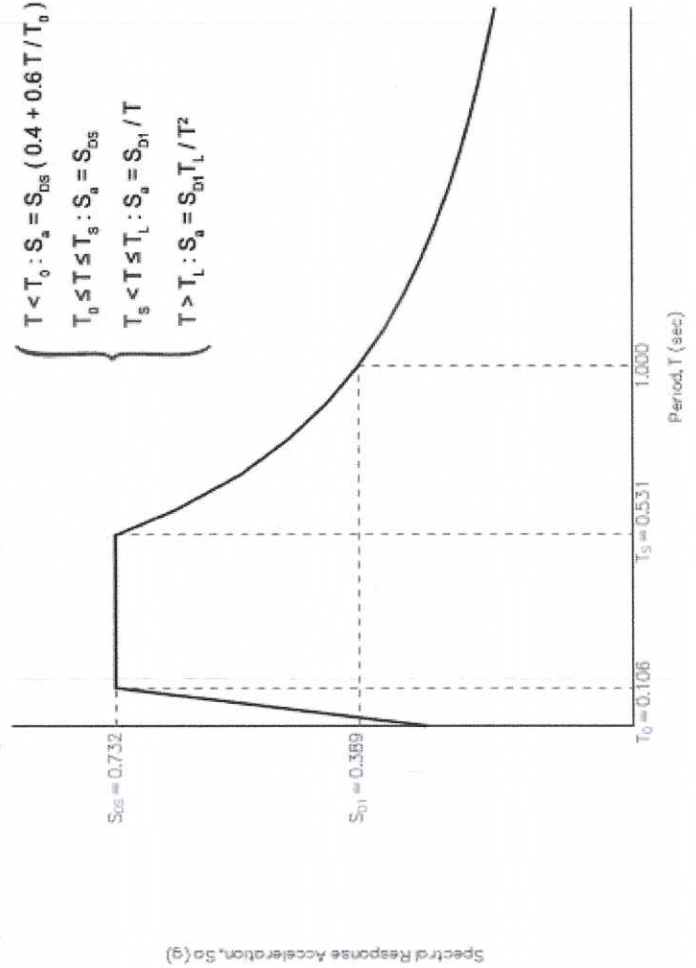
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.583 = 0.389 \text{ g}$$

Section 11.4.5 — Design Response Spectrum

From [Figure 22-12](#) [3]

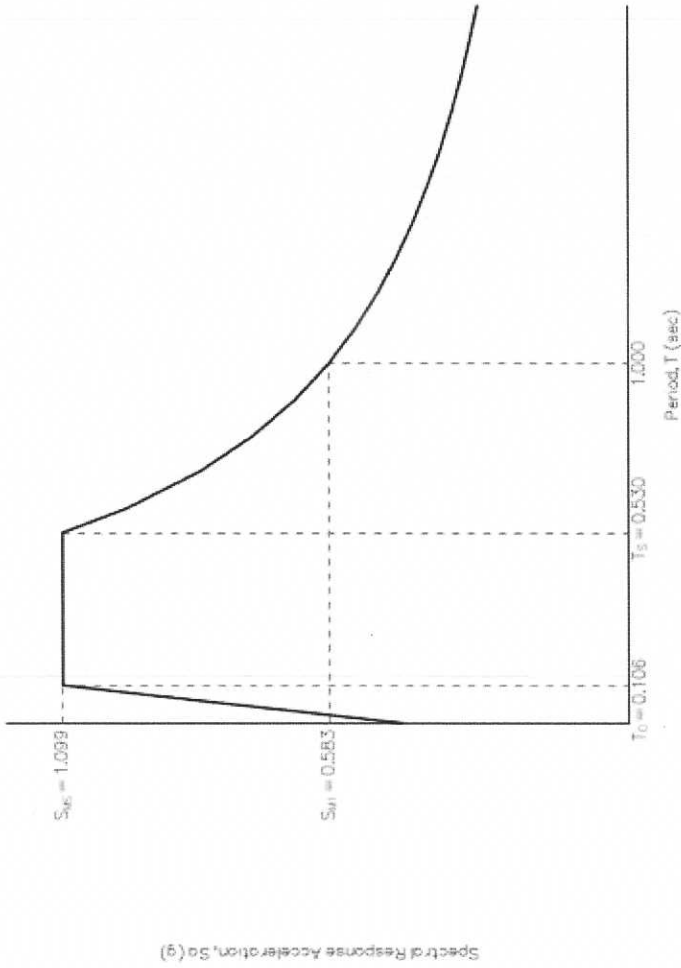
$$T_L = 8 \text{ seconds}$$

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From [Figure 22-7](#) [4]

$$PGA = 0.424$$

Equation (11.8-1):

$$PGA_M = F_{PGA}PGA = 1.000 \times 0.424 = 0.424 \text{ g}$$

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.424 g, $F_{PGA} = 1.000$

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From [Figure 22-17](#) [5]

$$C_{RS} = 0.972$$

From [Figure 22-18](#) [6]

$$C_{R1} = 1.022$$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and $S_{DS} = 0.732g$, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and $S_{D1} = 0.389g$, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category \equiv "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

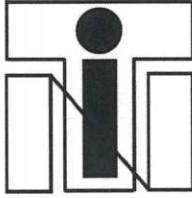
References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf

5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure-7_Figure_22-18.pdf

APPENDIX C

T.I.N. ENGINEERING REPORT



T.I.N. ENGINEERING COMPANY

Geotechnical • Structural • Environmental

17834 Bailey Drive • Torrance, CA 90504
Tel: (310) 371-7045, tinsoilsheep@gmail.com

File No.: 170040
September 29, 2017

Mr. Kenneth K. Wang
Metro Properties Group, LLC
4433 Canvoy Street, Suite H
San Diego, California 92111

SUBJECT: Permeability of On-Site Upper Soils for Proposed Car Wash at 2190 South El Camino Real,
Oceanside, California

Dear Mr. Wang:

In accordance with your request, we have completed this letter in order to provide the permeability of the on-site upper soils for the hydraulic designs at the subject site. On September 16, 2017 through September 18, 2017, two test pits were dug down to 48 inches deep. The test pits were 24 inches by 24 inches wide. The encountered upper 6 inch topsoil consisted of a dry and loose soil. Therefore, the bottom 42-inch deep soil in the test pits was tested for the soil permeability. The bottom soil consisted of a tan sandstone and claystone bedrock soil materials. The test pits were pre-soaked on September 16, 2017. Then, the test pits were tested for the soil permeability. On September 17, 2017, approximately 30 and 15 gallons of water were refill into the test pit Nos. 1 and 2, respectively, to 6 inches below the top of the test pits. It took approximately 24 hours to percolate water about 9 inches for the test pit No. 1 and 6 inches for the test pit No. 2 through the sides and bottom of the test pits.

Based upon the test results, the permeability of 0.00836 inch per minute for the test pit No. 1 and of 0.00418 inch per minute for the test pit No. 2 were obtained. Therefore, we recommend that the permeability of 0.00418 inch per minute may be used for the hydraulic design for the on-site upper soils.

Thank you for this opportunity to be of service. If you have any questions regarding this report, please contact the undersigned at the letterhead location.

Very truly yours,

T.I.N. ENGINEERING COMPANY



Tony S. C. Lee, M.S., P.E.
Project Engineer

TSCL:ir

Distribution: Client (1, by email)