

BEHAVIOR OF MANUFACTURED SAND UNDER CYCLIC LOADING

by

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ABSTRACT

BEHAVIOR OF MANUFACTURED SAND UNDER CYCLIC LOADING

The disposal or utilization of industrial wastes or by products in powder size is an important issue for all over the world. The fly ash, as one of the most significant waste of this industry, with millions of tons of production every year, poses problems such as handling, disposal and stability of the disposal sites due to its silt size etc. Manufacturing geomaterials using powder waste material has great potential for safe disposal and utilization when needed. To solve the growing disposal problems, fly ash has been pelletized to manufacture sand in two different gradations and its cyclic behavior has been evaluated in this thesis. In order to evaluate the performance of fly ash pellets; natural sand samples and fly ash pellets with similar size samples were prepared. The fly ash pellets have high porosity and low unit weight similar to those of calcareous sands. Due to lack of information of fly ash pellets, the literature related to the cyclic behavior of calcareous sand is investigated. A series of isotropically consolidated undrained monotonic triaxial tests are performed and internal friction angles are determined. Two quartzatic sands, poorly graded Kilyos Sand and well graded Sile Sand were used as control for comparison to manufactured fly ash pellets with similar gradations. For both natural and manufactured sand, samples were prepared for three different relative densities, loose to dense. The range of internal friction angles of sands studied was between 36° - 40° . For the manufactured sand the range was between 31° - 38° . The natural and manufactured sands were subjected to constant volume cyclic direct simple shear tests under 50 and 100kPa consolidation pressures. The cyclic stress ratios of 0.2 and 0.3 and cycle period of 1 and 2 were applied. The secant shear moduli and the damping ratio are calculated. These values are found to be similar to quartz sands.

ÖZET

İMAL EDİLMİŞ KUMLARIN TEKRARLI YÜKLER ALTINDA DAVRANIŞI

Toz boyutundaki ağır endüstriyel atıkların veya yan ürünlerin depolanması veya kullanımı tüm dünyada önemli bir meseledir. Her yıl milyonlarca ton üretimiyle bu endüstrinin en dikkate değer atığı olan uçucu kül, silt boyutundan dolayı, taşınma, depolama ve depolama sahalarının stabilitesi gibi problemler teşkil etmektedir. Toz boyutunda atık malzeme kullanarak geomalzeme üretimi ihtiyaç durumunda güvenli depolama ve kullanım için oldukça büyük bir potansiyele sahiptir. Bu tezde, artan depolama probleminin çözümü için, uçucu kül peletlenerek iki farklı granülometride kum imal edildi ve bunların dinamik davranış özellikleri incelendi. Uçucu kül peletlerinin performansının değerlendirilmesi için, aynı granülometride doğal kum ve uçucu kül peletler numuneleri hazırlandı. Uçucu kül peletler yüksek porozite ve düşük birim hacim aşırılığa sahip olduğundan kalkerli kumlara benzerlik göstermektedirler. Uçucu kül peletlere dair bilgi eksikliğinden dolayı, kalkerli kumların dinamik davranışına dair literatür incelendi. Isotropik gerilme şartlarında konsolidasyonlu ve drenajsiz olarak bir set monotonik üç eksenli deneyleri gerçekleştirildi ve iç sürtünme açışları belirlendi. Kontrol olarak, aynı granülometriye sahip uçucu kül peletlerle karşılaştırmak için iki kuvars kum, zayıf gradasyonlu Kilyos Kumu ve iyi gradasyonlu Şile Kumu kullanıldı. Hem doğal hem imal edilmiş kum için, gevşekten sıkıya olacak şekilde üç farklı izafi sıklıkta numuneler hazırlandı. Bulunan iç sürtünme açıları kumlar için 36° ila 40° arası bulundu. İmal edilmiş kumlar için bu aralık 31° ila 38° arası bulundu. Doğal ve imal edilmiş kumlar 50 ve 100 kPa konsolidasyon basıncı altında sabit hakim dinamik direct basit kesme deneylerine tabi tutuldu. 0.2 ve 0.3 dinamik basınç oranları ve 1 ve 2 saniye çevrim periyodları uygulandı. Sekant kesme modülleri ve sönümlleme oranları hesaplandı. Bu değerler kuvars kumlara benzer bulundu.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	iii
ABSTRACT	iv
ÖZET	v
LIST OF FIGURES	ix
LIST OF TABLES	xxi
LIST OF SYMBOLS	xxii
LIST OF ACRONYMS/ABBREVIATIONS	xxiv
1. INTRODUCTION	1
2. LITERATURE REVIEW	3
2.1. The Quartz Sands	3
2.2. Calcareous Sands	4
2.2.1. Properties and Mechanical Behavior of Calcareous Sands	6
2.3. Fly Ash Pellets	16
2.3.1. Fly Ash	16
2.3.2. Properties of Fly Ash	20
2.3.3. Pelletization	20
2.3.4. Manufacturing of Pellets with Fly Ash	22
2.3.5. The Crushing Behavior of fly Ash Pellets	22
2.4. Particle Shape Quantification and Image Processing	23
3. METHODOLOGY	26
3.1. Materials	26
3.1.1. Natural Sands	26
3.1.2. Fly Ash Pellets	28
3.1.2.1. Pelletization Theory	28
3.1.2.2. Fly Ash	30
3.1.2.3. Pelletization Process	30
3.2. Experimental Setups and Procedure	32
3.2.1. The Shape Factor Analysis	32
3.2.2. Crushing Strength Test	36

3.2.3.	Cyclic Direct Simple Shear Test	37
3.2.4.	Summary of ASTM Testing Requirements for Direct Simple Testing	40
3.2.5.	Constant Volume / Constant Load Tests	41
3.2.6.	Experiment Procedure	45
3.3.	Evaluation of Data	48
3.3.1.	Damping Ratio	50
3.3.1.1.	Calculation of Damping Ratio	50
3.3.1.2.	Calculating Damping Ratio by Using Cyclic Simple Shear Test Results	53
3.3.1.3.	Finding the Area Inside of the Hysteresis Loop	53
3.3.2.	Secant Modulus	54
3.3.3.	Triaxial Shear Test	55
3.3.3.1.	Triaxial Testing Sample Preparation	57
3.3.3.2.	Triaxial Testing Procedure	57
4.	RESULTS	59
4.1.	Shape Factor Analysis	59
4.2.	Crushing Strength Test	60
4.3.	Monotonic Triaxial Test Results	61
4.4.	Secant shear modulus (G_{sec}) and normalized shear modulus (G_{sec}/G_{max})	61
4.4.1.	The Results for 50 kPa Consolidation Pressure	62
4.4.1.1.	Sile Sand and the Fly Ash Pellet Sample with Same Gradation	62
4.4.1.2.	Kilyos Sand and Fly Ash Pellets with Same Gradation	64
4.4.2.	The Results for 100 kPa Consolidation Pressure	66
4.4.2.1.	Sile Sand and Fly Ash Pellets with Same Gradation . .	67
4.4.2.2.	Kilyos Sand and Fly Ash Pellets with Same Gradation	71
4.5.	Damping Ratio	75
4.5.1.	The Results for 50 kPa Consolidation Pressure	76
4.5.1.1.	Sile Sand and the Fly Ash Pellets with the Same Gra- dation	76
4.5.1.2.	Kilyos Sand and the Fly Ash Pellets with Same Gradation	77
4.5.2.	The Results for 100 kPa Consolidation Pressure	79

4.5.2.1. Sil Sand and the Fly Ash Pellets with the Same Gradation	79
4.5.2.2. Kilyos Sand and the Fly Ash Pellets with Same Gradation	81
4.5.3. The Variation of Damping Ratios Versus Relative Density . . .	83
4.5.4. The Variation of Number of Cycles to Arrive 0.1 Strain Level .	86
4.5.5. G_{sec} values corresponding different strain values	88
4.5.6. The Effect of Void Ratio to the Shear Modulus Values	94
5. SUMMARY	97
6. CONCLUSIONS	100
APPENDIX A: THE SHEAR SECANT MODULUS AND DAMPING RATIO	
CURVES FOR ALL THE STUDIES DONE	101
REFERENCES	128

LIST OF FIGURES

Figure 2.1.	Typical Shearing Responses of Granular Soils (Craig, 2004).	4
Figure 2.2.	Photographs of Calcareous Soil Particles.	7
Figure 2.3.	Tampa Bay Sand Direct Simple Shear Drained Strength Test (Brandes, 2011).	8
Figure 2.4.	Kawaihae Sand Direct Simple Shear Drained Strength Test (Brandes, 2011).	9
Figure 2.5.	Full Scale Pelletizer (Baykal, 2015).	17
Figure 2.6.	SEM of Typical Fly Ash Particles (Danyıldız, 2007).	18
Figure 2.7.	Formation of Pellet Particles.	21
Figure 2.8.	Qualitative Sphericity-Roundness Chart (Powers, 1953).	24
Figure 2.9.	Comparison Chart for Roundness (Krumbein, 1941).	24
Figure 2.10.	Particle Roundness- Sphericity Chart (Krumbein and Sloss, 1963).	25
Figure 3.1.	Kilyos Sand Size Distribution (Ranjpour, 2015).	27
Figure 3.2.	Sile Sand Size Distribution (Ranjpour, 2015).	27
Figure 3.3.	Mechanism of Pellet Formation (Jaroslav and Ruzickova, 1988).	28
Figure 3.4.	Sketch of Pelletization Disc Unit (back view) (Döven, 1998).	29

Figure 3.5.	Sketch of Scrapping Pellets Positioning (Erdurak,2011).	31
Figure 3.6.	The Grain Size Evolution of fly Ash Pellets.	32
Figure 3.7.	Example Image of Some Sand Particles.	33
Figure 3.8.	Example Binary Image.	34
Figure 3.9.	Example Image of Numbers Ascribed to Sand Particles.	34
Figure 3.10.	Example ImageJ Output Table.	34
Figure 3.11.	The Crushing Strength Test (Döven, 1998).	36
Figure 3.12.	SGI Simple Shear Device 1936 (Kjellam, 1951).	38
Figure 3.13.	Direct and Simple Shear Comparison.	38
Figure 3.14.	GEOCOMP Direct Simple Shear Test Device.	39
Figure 3.15.	Schematic Diagram of Essential Direct Simple Shear Components (ASTM D6528).	41
Figure 3.16.	Samples Prepared for DSS Testing Using Stacked Rings.	42
Figure 3.17.	The Effects of Different Confining Systems (GEOCOMP DSS test manual 2012).	43
Figure 3.18.	GeoComp Direct Simple Shear Apparatus (GEOCOMP DSS test manual 2012).	44

Figure 3.19. The Mechanism that Apply the Loads on the Cyclic Direct Simple Shear Test apparatus.	45
Figure 3.20. Direct Simple Shear Confinement Types: wire reinforced membranes (left) and stacked rings (right) (Baxter <i>et al.</i> , 2010).	46
Figure 3.21. An Ideal Waveform and the Limit Value of 10% Difference for Two Successive Half-Cycles (Silver,1977).	47
Figure 3.22. The Limit of 10% Difference in Duration of Successive Half-Cycles and the Unacceptable Waveform Due to Spikes at Peaks (Silver, 1977).	48
Figure 3.23. Typical Report for Consolidation Phase.	49
Figure 3.24. Typical Report for Shearing Phase.	49
Figure 3.25. Typical Strain Ranges and tests for These Ranges.	50
Figure 3.26. Use of Hysteresis Loop for Calculation of Damping Ratio (Kramer, 1996).	51
Figure 3.27. Damping Ratio versus G/G_{max} for sands (Khoury, 1984).	52
Figure 3.28. Shear Modulus Reduction and Equivalent Damping Ratio Curves (Kramer, 1996).	52
Figure 3.29. The Selection of Curve Fitting Alternatives.	53
Figure 3.30. The Mathematical Background of Finding the Area Under a Curve.	54
Figure 3.31. Typical Hysteresis Loop (Kramer, 1996).	55

Figure 3.32.	Fully Automated Triaxial Testing Apparatus (Abatkon, 2012).	56
Figure 3.33.	Schematic Representation of Flow Trac Unit (Abadkon, 2012).	57
Figure 4.1.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CRS=0.2).	62
Figure 4.2.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).	63
Figure 4.3.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=2 CSR=0.2).	64
Figure 4.4.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.2).	65
Figure 4.5.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).	65
Figure 4.6.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=2 CSR=0.2).	66
Figure 4.7.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.2).	68
Figure 4.8.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).	68
Figure 4.9.	Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions ((CP=2 CSR=0.2).	69

Figure 4.10. Grain Size Analysis After and Before the Test at CSR=0.2 and CP=1for Sile Sand.	69
Figure 4.11. Grain Size Analysis After and Before the Test at CSR=0.3 and CP=1for Sile Sand.	70
Figure 4.12. Grain Size Analysis After and Before the Test at CSR=0,3 and CP=1for Well Graded FA Pellets.	71
Figure 4.13. Grain Size Analysis After and Before the Test at CSR=0,2 and CP=1for Kilyos Sand.	72
Figure 4.14. Grain Size Analysis After and Before the Test at CSR=0,3 and CP=1for Kilyos Sand.	72
Figure 4.15. Grain Size Analysis After and Before the Tests for Kilyos Sand. . .	73
Figure 4.16. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2). . .	74
Figure 4.17. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	74
Figure 4.18. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2). . .	75
Figure 4.19. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2). . .	76
Figure 4.20. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	77

Figure 4.21. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2). . .	77
Figure 4.22. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	78
Figure 4.23. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	78
Figure 4.24. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2). . .	79
Figure 4.25. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2). . .	80
Figure 4.26. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	80
Figure 4.27. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2). . .	81
Figure 4.28. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2). . .	82
Figure 4.29. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3). . .	83
Figure 4.30. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2). . .	83
Figure 4.31. The Effect of Relative Density on Damping Ratio for Sile Sand and Well Graded Fly Ash Pellets.	84

Figure 4.32. The Effect of Relative Density on Damping Ratio for Kilyos Sand under Given Loading Conditions.	85
Figure 4.33. The Effect of Relative Density on Damping Ratio for Kilyos Sand and Poorly Graded Fly Ash Pellets.	86
Figure 4.34. The Number of Cycles to Arrive 0.1 Strain.	87
Figure 4.35. The Number of Cycles to Arrive 0.1 Strain.	87
Figure 4.36. The Number of Cycles to Arrive 0.1 Strain.	87
Figure 4.37. The Number of Cycles to Arrive 0.1 Strain.	88
Figure 4.38. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=1 Loading Conditions.	88
Figure 4.39. Shear Moduli at Different Shear Strain Rate for CSR=0.3 and CP=1 Loading Conditions.	90
Figure 4.40. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=2 Loading Conditions.	91
Figure 4.41. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=1 Loading Conditions.	92
Figure 4.42. Shear Moduli at Different Shear Strain Rate for CSR=0.3 and CP=1 Loading Conditions.	93
Figure 4.43. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=2 Loading Conditions.	94

Figure 4.44. Secant Shear Modulus for 0.001, 0.01 and 0.1 Shear Values at Different Void Ratio Values under 100 kPa Consolidation Pressure. . .	95
Figure 4.45. Secant Shear Modulus for 0.001, 0.01 and 0.1 Shear Values at Different Void Ratio Values under 50 kPa Consolidation Pressure. . .	96
Figure A.1. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	101
Figure A.2. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	101
Figure A.3. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions for Well Graded Pellet Sample. . . .	102
Figure A.4. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.	103
Figure A.5. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.	103
Figure A.6. Normalized Shear Modulus Reduction Curves for well Graded Fly Ash Pellets at Given Loading Conditions.	104
Figure A.7. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.	104
Figure A.8. Shear Modulus Reduction Curves for well Graded Fly Ash Pellets at Given Loading Conditions.	105
Figure A.9. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	105

Figure A.10. Normalized Shear Modulus Reduction Curve for Kilyos Sand at Given Loading Conditions.	106
Figure A.11. Shear Modulus Reduction Curvesfor Poorly Graded Fly Ash Pellets at Given Loading Conditions.	106
Figure A.12. Normalized Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.	106
Figure A.13. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	107
Figure A.14. Normalized Shear Modulus Reduction Curve for Kilyos Sand at Given Loading Conditions.	107
Figure A.15. Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pel- lets at Given Loading Conditions.	108
Figure A.16. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	109
Figure A.17. Shear Modulus Reduction Curves for Poorly Graded at Given Load- ing Conditions.	110
Figure A.18. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	110
Figure A.19. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	111
Figure A.20. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.	111

Figure A.21. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	112
Figure A.22. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.	112
Figure A.23. Shear Modulus Reduction Curve for Well Graded Fly Ash Pellets at Given Loading Conditions.	112
Figure A.24. Normalized Shear Modulus Reduction Curve for well Graded Fly Ash Pellets at Given Loading Conditions.	113
Figure A.25. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.	113
Figure A.26. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.	114
Figure A.27. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	115
Figure A.28. Shear Modulus Reduction Curves for Poorly Graded Pellets at Given Loading Conditions.	116
Figure A.29. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	117
Figure A.30. Shear Modulus Reduction Curves for Poorly Graded Pellets at Given Loading Conditions.	118
Figure A.31. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.	119

Figure A.32. Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.	120
Figure A.33. Damping Ratio Values for Şile Sand at Given Loading Conditions.	120
Figure A.34. Damping Ratio Values for Well Graded Fly Ash Pellets at Given Loading Conditions.	121
Figure A.35. Damping Ratio Values for Şile Sand at Given Loading Conditions.	121
Figure A.36. Damping Ratio Values Forwell Graded Fly Ash Pellets at Given Loading Conditions.	121
Figure A.37. Damping Ratio Values for Şile Sand At Given Loading Conditions.	121
Figure A.38. Damping Ratio Values For Well Graded Fly Ash Pellets at Given Loading Conditions.	122
Figure A.39. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 1.	122
Figure A.40. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 2.	123
Figure A.41. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 3.	123
Figure A.42. Damping Ratio Values for Şile Sand at Given Loading Conditions.	124
Figure A.43. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.	124

Figure A.44. Damping Ratio Values for Şile Sand at Given Loading Conditions.	124
Figure A.45. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.	125
Figure A.46. Damping Ratio Values for Şile Sand at Given Loading Conditions.	125
Figure A.47. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.	125
Figure A.48. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.	126
Figure A.49. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.	126
Figure A.50. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.	126
Figure A.51. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.	126
Figure A.52. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.	127
Figure A.53. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.	127

LIST OF TABLES

Table 2.1.	A Brief Summary of The Studieis About the Shear Properties of the Calceraous Sands.	11
Table 2.2.	The Crushing Strength Values for Different Curing Time (Danyıldız, 2007).	23
Table 3.1.	Properties of Sile and Kilyos Sands.	27
Table 3.2.	Maximum and Minimum Void Ratios.	29
Table 3.3.	Physical and Chemical Characteristics of Fly Ash (Saygili, 2008). .	30
Table 3.4.	Comparison of Theoretical and Predicted Shape Parameter Values.	33
Table 3.5.	Number of Analyzed Particles (Cox,2008).	35
Table 3.6.	Cyclic DSS Tests CSR and CP Combinations.	46
Table 4.1.	Average Sphericity and Roundness Values for All Sand Types. . . .	59
Table 4.2.	Crushing Strength Test Results.	60
Table 4.3.	Internal Friction Angles of Samples.	61

LIST OF SYMBOLS

A_c	Corrected area of the sample
A_0	Initial area of the sample
A_m	Area of the membrane
B	Pore pressure parameter
C_{comp}	Compression index of the soil
C_u	Coefficient of uniformity
C_c	Coefficient of curvature
D_R	Relative density
D_{50}	Median grain size
$d_{\varepsilon v}$	Volumetric strain change
$d_{\varepsilon q}$	Deviatoric strain change
E_m	Young's modulus for the membrane material
e_{min}	Minimum void ratio
e_{max}	Maximum void ratio
e	Void ratio
F_b	Buoyancy force
h	Sand raining height
c	Height of sample after consolidation
h_0	Initial height of the sample
I_d	Density index
I_R	Relative density index
K	Coefficient of the internal friction
L	Unstretched length of the membrane
OCR	Overconsolidation ratio
P'_A	Atmospheric pressure
p'	Mean effective stress
q	Deviatoric stress
r	Constant parameter showing the amount of dilatancy contribution to shear strength property of soil

t_m	Membrane thickness
V_c	Volume of the sample after consolidation phase
W_s	Width of circumferential strip
α	Angle of shearing in sawtooth plane
α	Fitting parameter depending on D_{50}
Δu	Pore pressure change
ΔL	Change in length of the membrane
Δh	Change in height of the specimen
ε_a	Axial strain
ε_q	Shear strain
ε_r	Radial strain
ε_v	Volumetric strain
$\dot{\varepsilon}_1$	Strain rate in vertical direction
$\dot{\varepsilon}_2$	Strain rate in horizontal direction
$\dot{\varepsilon}_3$	Strain rate in horizontal direction
$\dot{\varepsilon}_v$	Volumetric strain rate
$\dot{\varepsilon}$	Shearing strain rate for triaxial test
ϕ'	Friction angle
λ	Ratio of the effective horizontal stress change to effective vertical stress change
σ'_{1f}	Effective vertical stress at failure
σ'_{3f}	Effective horizontal stress at failure
σ'_c	Effective confining pressure
σ'_n	Effective normal stress
τ	Shear stress

LIST OF ACRONYMS/ABBREVIATIONS

ASTM	American Society for Testing and Materials
CD	Consolidated Drained
FEM	Finite Element Method
DSS	Direct Simple Shear
CP	Cyclic Period
CSR	Cyclic Stress Ratio
CSS	Cyclic Simple Shear
FS	Factor of Safety
LM	Light Microscope
SGI	Swedish Geotechnical Institute

1. INTRODUCTION

The research presented in this thesis is an effort to better investigate the dynamic properties of fly ash pellets. They are proposed as a new method for facilitating the disposal of fly ash which is the waste material of pulverized coal burning factories and power plants. In this scope, regarding their similarity of behavior with calcareous sands, in the literature review the works done with the calcareous sands will be investigated and explained. The calcareous sands are the closest natural material to the fly ash pellets. This provides a good base of information on the way to determine the characteristic of this newly proposed method and the material.

In this thesis a light microscope and a digital microscope camera (Mshot) were used to capture the particle images and an image processing software (Image J) was used to process the images, to obtain sphericity and roundness for each grain. All the index properties of the fly ash pellets, which are produced in Karl Terzaghi Soil Mechanics Laboratory, as well as Kilyos and Sile sand, are investigated. Furthermore Consolidated-Drained (CD) Triaxial tests were performed on 2 different sand types and 2 different fly ash pellet sample in order to define the basic parameters to define the strength characteristics of fly ash pellets and to compare them with the same graded sand samples. Also, Constant Volume Cyclic Simple Shear tests were conducted to evaluate some characteristic under cyclic loading such as shear modulus and damping ratio of fly ash pellets.

Calcareous sands are involved and used as construction material in a large number of artificial islands and harbor construction/expansion projects. An improved understanding of the liquefaction susceptibility of the aforementioned biogenic sediments is the main goal of this study. This will provide useful information regarding crucial aspects such as the necessity of attaining the highest degree of compaction which involves high costs, being over conservative and non-environmental friendly.

Their behavior under cyclic loading is defined by two contradictory aspects, their compressibility and angular particle nature. The evaluation of cyclic behavior of crushable sands involves better understanding of these aspects, and their relation.

The crushable sand's high compressibility is associated with its mineralogy, porous nature of particles, and generally high void ratios. Their crushable nature leads to a significant rearrangement of the grains during cyclic loading. Comprehending the influence of high compressibility to the generation of pore water pressures (volume change) is very important for evaluating their cyclic behavior. Generally, when the volume of the soil decreases rapidly (crushable nature) pore water pressures increase dramatically, thus the cyclic shear strength is decreased. On the other hand, the angular shape of the crushable sand grains contributes to a more stable interlocking of the grains during cyclic shearing caused by an earthquake excitation. This property is considered to ameliorate the cyclic behavior of carbonate sands (Coop and Airey, 2003).

Secondly, even though the studies on earthquakes dates back for many centuries, the analysis of earthquake effect on structures based on the soil behavior is a very recent. The nature and distribution of earthquake damage is strongly affected by response of soil deposits and earth structures under seismic loading conditions. The evaluation of the dynamic properties of natural soil deposits is of prior importance in order to solve problems involving soil-structure interactions, to study the site response, to predict the ground motion and to proceed to seismic zonation. Ground motion under earthquake loading is influenced by the soil condition, but the non-linearity of the soil behavior makes it difficult to estimate the site response. The values of dynamic properties of soils; shear modulus and damping ratio are influenced by plasticity index, void ratio, relative density, number of cycles, grain size distribution, type of soils, over burden pressure, location of ground water and amplitude of earthquake (amplitude of cyclic loading). The shear modulus and damping ratio are obtained from laboratory and field tests. In literature review basic dynamic properties of soils and soil response under cyclic loading are examined.

2. LITERATURE REVIEW

In this chapter, the calcareous sands will be studied due to uniqueness of this work as fly ash pellets are a newly proposed geotechnical material. On the other hand, the calcareous sands are a good starting point for this new research of fly ash pellets, as they are the material which has many similarities with fly ash pellets.

2.1. The Quartz Sands

The shear strength (τ_f) of a soil at a point on a particular plane was defined by Coulomb as a linear function of the normal stress at failure (σ_f) at the same plane and on the same point:

$$\tau_f' = \sigma_f' \tan \phi' \quad (2.1)$$

Where τ_f' is the shear stress, σ_f' is the effective normal stress, and ϕ' is the effective friction angle. However the linear failure envelope represents the soil at critical state conditions. The critical state concept was proposed by Roscoe, et al. (1958) and Schofield and Wroth (1968), and is defined as the end or ultimate state of a deformation process, in which soil continues to shear under constant stress without volume change. Besides, critical state friction angle, which is constant for a given sand and depends only on the mineralogy of the sand grains, is defined as the friction angle of the soil shearing at a constant volume.

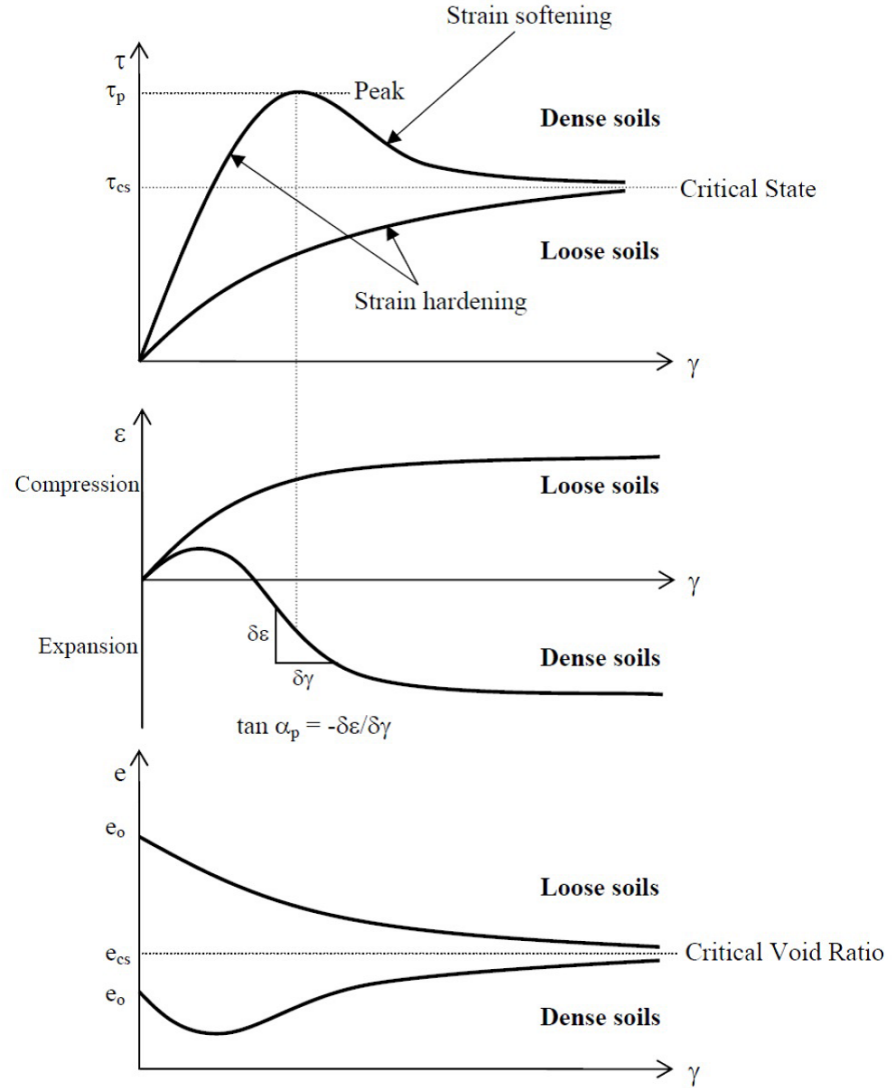


Figure 2.1. Typical Shearing Responses of Granular Soils (Craig, 2004).

2.2. Calcareous Sands

Starting from 60s, especially in offshore construction driven by the petroleum industry encountered calcareous sediments in the Arabian Gulf and since then in many other world areas. Holmes stated that calcareous sediments cover approximately 40 % of the ocean floor (Holmes, 1978). Some places which has calcareous sand deposits poses various problems and becomes challenging for the construction as the behavior of calcareous sands is different and sometimes needs a different approach. This mis-

understanding of behavior can be concluded as a perilous, where considerable amount of source is wasted and, moreover, great loss of human life can be a result.

A variety of soft carbonate grains cemented with precipitated calcite form a porous structure that experiences diagenetic changes after deposition. The resultant soil structure responds to load in a brittle manner, tends to contract during shear, is very susceptible to cyclic strength degradation, is highly compressible under bearing pressures. Still remaining to be determined, in support of general design procedures for foundations on such materials, are: relevant parameters, means to quantify the parameters, and methods to apply them. For the present, design is largely empirical and dependent upon site - specific, full - scale field load tests (Holmes, 1978; McClelland, 1988).

They are especially widespread in tropical and subtropical shallow waters of the Pacific Ocean where seismic hazards are often very high. The potential for damages caused by strong shaking of calcareous sediments has been made painfully clear by numerous catastrophic earthquakes such as the Guam earthquake of 1993, the Hawaii earthquake of 2006, the Haiti earthquake of 2010 and many others. In each of these cases, calcareous sediments were observed to liquefy and spread, causing extensive damage to port facilities and other offshore structures. It is well known that calcareous sediments pose a challenge to offshore construction, particularly the installation of driven piles and their response to dynamic loads (McClelland, 1988).

Even that the character of biogenic calcareous sands is quite different from that of quartz sands, our knowledge of the dynamic behavior of granular soils comes primarily from studies conducted on quartz sands of terrigenous origin. Calcareous sands typically have lower grain hardness, larger intergranular porosity, a wider range of grain shapes, more complex structural arrangements that reflect unique post-depositional processes of cementation, dissolution, recrystallization, and other diagenetic changes.

It is not surprising then that there are significant differences in mechanical behavior among various types of calcareous sands, and with respect to quartz sands

(Celestino and Mitchell, 1983).

Field and laboratory investigations have highlighted differences in terms of compressibility, volume changes and grain crushing during shearing, yielding, friction, and water permeation (Datta *et al.*, 1979; Semple, 1988; Al-Douri and Poulos, 1991; Jewell, 1993; Kwag *et al.*, 1999; Ong *et al.*, 1999; Coop and Airey, 2003). Although there have been a few studies that have addressed the dynamic behavior of calcareous sediments (Lo Presti *et al.*, 1993; Kwag *et al.*, 1999; Sharma and Fahey, 2003; a,b, Mao and Fahey, 2003; Seidman, 2007; Porcino *et al.*, 2008), most of them have not sought to draw particular distinctions with the corresponding behavior of terrigenous soils (Datta *et al.*, 1979; Semple, 1988; Al-Douri and Poulos, 1991; Jewell, 1993; Kwag *et al.*, 1999; Ong *et al.*, 1999; Coop and Airey, 2003; Lo Presti *et al.*, 1993; Kwag, *et al.*, 1999; Sharma and Fahey, 2003; a,b, Mao and Fahey, 2003; Seidman, 2007; Porcino *et al.*, 2008).

What evidence exists is limited and sometimes contradictory. Some laboratory studies indicate that the cyclic strength of loose calcareous sands is higher than that of quartz sands at equivalent densities and stress conditions (Chen, Morioka and Nicholson, Nicholson). Rollins *et al.*, suggest that liquefaction is triggered at different stress levels in reefoidal calcareous sand deposits compared to quartz sand deposits, again suggesting that cyclic strengths are different. On the other hand, Mejia and Yeung report similar liquefaction behavior for both classes of sands based on observations of field liquefaction (Chen 1985; Morioka and Nicholson, 2000; Nicholson, 2006; Rollins *et al.*, 2004; Mejia and Yeung, 1995).

2.2.1. Properties and Mechanical Behavior of Calcareous Sands

Calcareous soils are formed as a result of chemical deposition or physical weathering. During chemical deposition, calcium carbonate comes out of solution as a precipitate in sea water. Physical weathering breaks down the skeletal remains of marine organisms, such as mollusks, foraminifera, corals and algae into sand-sized particles (Coop and Airey, 2003). The sands used for this research consist primarily of physi-

cally weathered mollusk shells and coral fragments. The marine organisms that create the source material for these sands are prevalent in warm, shallow tropical and subtropical shelves (Golightly and Hyde, 1988). In the United States these areas include Guam, Hawaii, Puerto Rico, and parts of the Gulf Coast (Morioka and Nicholson, 2000). Due to the nature of their origin, calcareous sand particles have two attributes that cause their properties to differ from those of quartzitic soils. Calcareous sands have significant intra-particle void space (Coop and Airey, 2003; Datta *et al.*, 1982; Golightly and Hyde, 1988; Hyodo *et al.*, 1996; Hyodo *et al.*, 1998; Nicholson, 2006; Sharma and Ismail, 2006). The intra-particle void space is created by the particles made of shells that have not been fully broken apart or by corals that have cavities within the particle or on the surface. Photographs of a variety of calcareous soil particles are shown in Figure 2.2.

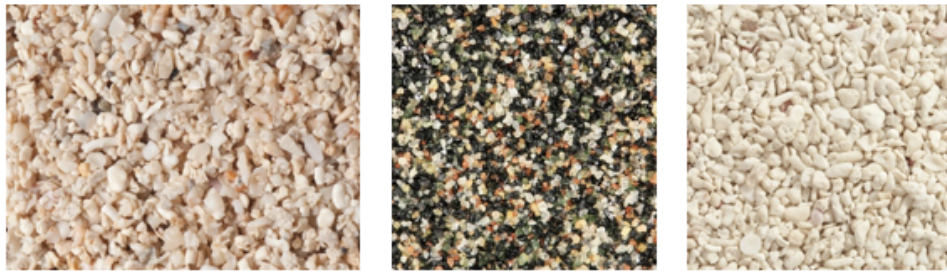


Figure 2.2. Photographs of Calcareous Soil Particles.

The second attribute that affects the behavior of these sands is the angular particle shape. There are a variety of particle shapes encountered in calcareous soils. Curved flat particles come from fragments of small shells. Hollow tube-shaped particles are the exoskeletons of other small mollusks. A loose, complex interlocking structure is created by these angular soil particles. This in combination with the intra-particle pore space creates a very high void ratio. Void ratio in excess of 1.0 is commonly seen in calcareous soils, (Coop and Airey, 2003; Datta *et al.*, 1982; Golightly and Hyde, 1988; Hyodo *et al.*, 1996; Hyodo *et al.*, 1998; Nicholson, 2006; Sharma and Ismail, 2006). Properties of sands vary significantly in quartzitic sands, most engineering properties can be related to the grain size distribution and relative density (Mitchell, 1993). This is not the case with calcareous sands. (Coop and Airey, 2003) The most significant

factors affecting engineering properties of calcareous sands are cementation and soil structure. The engineering properties of Playa Santa and other calcareous sands are discussed below.

For a brief and clear comparison of quartzitic sands and calcareous sands, in Figure 2.3 and Figure 2.4, Tampa Bay and Kawaihae Sand are compared. Tampa Bay Sand, is chemically composed of mainly quartz and the behavior we obtain by direct simple shear sand is general behavior of sands that are in loose condition (Brandes, 2011).

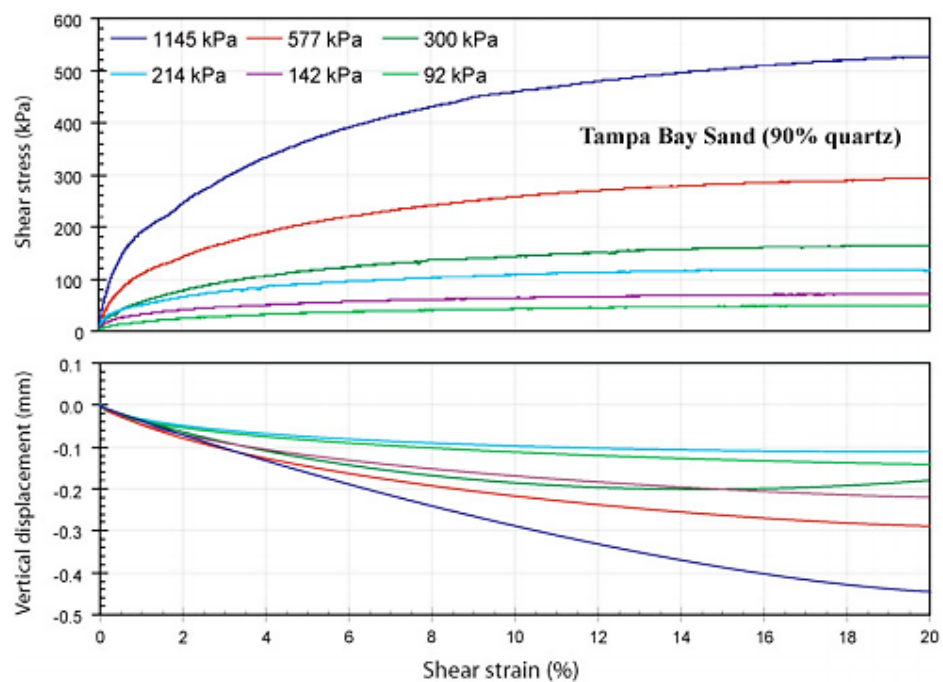


Figure 2.3. Tampa Bay Sand Direct Simple Shear Drained Strength Test (Brandes, 2011).

Again in the Direct Simple Shear tests, Kawaihae Harbor Sand, which has 100 % carbonate content, behaves significantly different than quartz sand. Even in the loose state, a dilation behavior can be seen and this behavior occurs at the large deformations (Brandes, 2011).

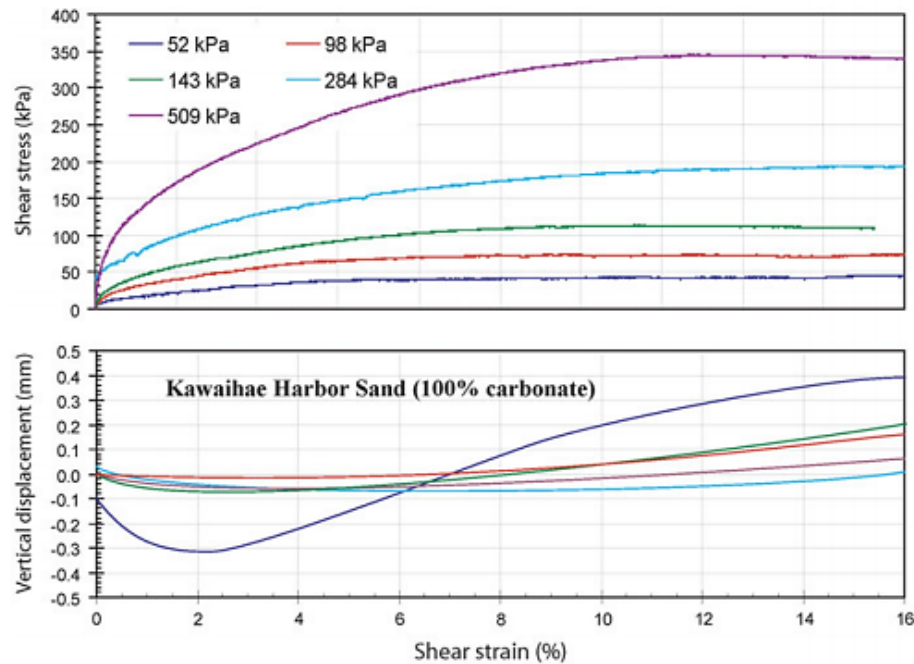


Figure 2.4. Kawaihae Sand Direct Simple Shear Drained Strength Test (Brandes, 2011).

The studies on calcareous sands are motivated by the offshore platforms when the first encounter with these materials happened. In 1982, Poulos, Uesugi and Young did a very early study about the strength and deformation properties of Bass Strait carbonate sands. The research included triaxial tests and their findings are presented. They found that increasing confining pressure reduced the friction angle and the dilative behavior. Then another study is done by Airey, Randolph, Hyden, (1988); with the same carbonate sand. This time, they proposed an empirical relationship between the mean effective stress and Young Modulus. Many researchers found that generally during drained shear, calcareous sands exhibit more strain during loading than quartzitic sands prepared at similar relative densities (Coop, 1990; Datta *et al.*, 1982; Demars and Chaney, 1982, Golightly and Hyde, 1988; Semple, 1988). They generally have greater peak strength friction angles, but this strength is not realized until significant strain has occurred. These soils are 1.5 to 2 times less stiff when compared to terrigenous soils (Morioka and Nicholson, 2000). Some researchers have reported strains as high as 20% before peak strength is reached (Sharma and Ismail, 2006).

During the 1988 Perth conference, a central theory about calcareous sands was discussed (Coop, 1988; Golightly and Hyde, 1988; Kaggwa *et al.*, 1988; Semple, 1988), and later formalized in more detail by Coop, (1990). This theory corresponds directly with the compressive behavior of calcareous materials discussed previously. Calcareous soils behave exactly as terrigenous soil would if the terrigenous soil had the same loose structure. The nature of the calcareous particles creates a soil fabric with a large of void ratio. It is not the calcareous minerals that govern the behavior of calcareous sands. Rather, it is the bulky soil fabric that governs the behavior (Coop, 1990). Past studies on the cyclic behavior of calcareous sands are limited. The cyclic behavior of calcareous soils was first researched by the petroleum industry for offshore oil platforms. During large storms the force of waves impacting the platforms creates a cyclic load. Researchers were interested in how foundations in calcareous sands would be affected. When compared to earthquake engineering, the shear stresses on these soils are relatively small. The number of cycles can be significant, in the range of several thousands (Airey *et al.*, 1988; Fahey, 1988; Kaggwa *et al.*, 1988; Sharma and Ismail, 2006). There are few studies on calcareous sands with a focus on earthquake loading. The most significant earthquake work on calcareous soils was performed by Japanese researchers Hyodo *et al.* (1996, 1998). Morioka and Nicholson, (1999, 2000) also performed a series of cyclic and monotonic triaxial tests and CPT calibration chamber testing. Table 2.1 is a detailed summary of studies on calcareous soils. Information is provided on the types of tests perform and the variables tested.

In all cases, studies that have compared the cyclic strength of calcareous sands to quartzitic sands have found calcareous sands to be stronger (Hyodo *et al.*, 1996; Hyodo *et al.*, 1998; Kaggwa *et al.*, 1988; Morioka and Nicholson, 1999). Authors have attributed this to the highly angular nature of calcareous sand particles. This angularity creates a more stable interlocking soil fabric, resistant to liquefaction. A brief summary of the studies that are conducted about the calcareous sands are presented in the Figure 2.1.

Table 2.1. A Brief Summary of The Studieis About the Shear Properties of the Calcreaous Sands.

Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
1982	Poulos, H. G. Uesugi, M Young, G. S.	Strength and deformation properties of Bass Strait carbonate sands	Offshore Platforms in Bass Strait	- Soil A, a gray sand with some silt Soil B, an orange-colored sand	CID triaxial comp.	- Increasing confining pressure reduced the friction angle . and the dilative behavior
1988	Airey, D. W. Randolph, M. F. Hyden, A. M.	The strength and stiffness of two calcreaous sands	Offshore Platforms in Bass Strait	- Kingfish, well graded sand - Halibut, medium to coarse grained sand	CID triaxial comp.	- Relatively high friction angle. Soil only dilate at low confining pressures. - Proposed an empirical relationship between the mean mean effective stress and Youngs Modulus (initial stiffness).
1988	Golightly, C.R. Hyde, A. F. L.	Some fundamental properties of carbonate sands	Offshore Platforms	-Leighton Buzzard sand, quartzitic sand - Dog Bay sand, molluscan carbonate sand, Ireland - Ballyconnely sand, a coralline algal carbonate sand, Ireland - Bombay Mix, a marine siliceous carbonate sand,India	CID and CIU triaxial comp.	- Stress-strain curves for drained tests showed calcreaous sand to be much less stiff than the quartzitic sand. Sands contract when sheared at relatively low effective confining pressures. - Found that friction angle of calcreaous sands are greater. Increased mineral friction angle component, with some decrease in the sliding, crushing, rearrangement component. (As described by Rowe stress dilatancy theory.) - Significant early generation of high positive excess pore pressure was observed in undrained tests.

Table 2.1. A Brief Summary of The Studieis About the Shear Properties of the Calceraous Sands.

Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
1988	Sample, R. M	The mechanical properties of carbonate soils	Offshore Platforms in Bass Strait	- Several Bioclastic calcareous soils from the Bass Strait	None	- Friction angles are greater in calcareous soils. Revisited Golightly's and Hyde's discussion of Rowe's stress dilatancy theory. Increased mineral friction with reduced dilatancy due to crushing. - Restated commonly accepted decreasing friction angle with increasing confining pressure. This change occurs at lower confining pressures in calcareous sands. - Suggests that it is the void ratio and particle angularity that govern the behavior not the carbonate minerals, i.e., if a quartzitic sand were tested with the same void ratio as calcareous sands the behavior would be comparable. Detailed discussion/argument for this by comparing very loose quartzitic sands to dense calcareous sands, equal void ratios. His argument is not well supported and makes no strong conclusions.
1990	Coop, M. R.	The mechanics of uncemented carbonate sands	Offshore Platform	- Dogs Bay calcareous sand	CID, CIU, CKoU triaxial comp.	- Higher friction angle in calcareous soils. Greater strains are needed to mobilize the frictional strength. High compressibility reduces the horizontal loads negatively affecting skin friction in piles. - The significance of compressibility to soil strength is dependent on the location of the in-situ stress-volume relative to the normal compression line. Coop makes this is an extremely insightful and underappreciated point.

Table 2.1. A Brief Summary of The Studieis About the Shear Properties of the Calcreaous Sands.

Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
1996	Hyodo, M. Aramaki, N. Itoh, M. Hyde, A. F. L.	Cyclic strength and deformation of crushable carbonate sand	Liquefaction susceptibility determination of crushable calcareous soils	-Dogs Bay calcareous sand	CIU and CAU triaxial comp. and exten.. Cyclic CU triaxial	- The analysis of the monotonic results is limited. A better review is included in their following paper. (Hyodo <i>et al.</i> , 1998, See Below) - Identifies the phase transformation point and compares it at varied parameters. - Hyodo understands and conveys the behavior very well, but makes few useful big picture conclusions.
1998	Hyodo, M. Hiroshima Hyde, A. F. L. Aramaki, N. Nakata, Y. % exten.	Liquefaction CIU in crushable soils - Crushable soils are more	Liquefaction - When sheared in a dense state crushable susceptibility determination of soils	-Ube Masado, weathered granite, washed - Shirasu, weathered	triaxial comp. and crushable Cyclic CU triaxial	soils have stress paths which are similar to those of loose sands of a less crushable nature. Volcanic soil, 35
			soils	Non-Plastic Fines - Dogs Bay, Calcareous Sand	Cyclic CU triaxial	dependent on the effective confining pressure because these stresses are often beyond the isotropic compression yield stress. - Identifies the phase transformation point and compares it at varied parameters.

Table 2.1. A Brief Summary of The Studieis About the Shear Properties of the Calcearaous Sands.

Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
1999	Morioka, B. T. Nicholson, P.G	Evaluation of the static and cyclic properties of alcareous sand in a calibration chamber study	Liquefaction susceptibility determination using penetration tests	-Ewa Plains Calcareous SP, Oahu, Hawaii - Monterey Sand (Scalped)	CIU triaxial comp. and Cyclic CU triaxial	- Found significantly higher friction angle of calcareous sand at several relative densities compared to Monterey Sand. - It required twice the strain to mobilize the peak strength in the calcareous sand. This is due to significant contractive tendencies of calcareous sands. - Found significant fluctuations in deviator and volume change data. Suggests that this is due to particle rearrangement and crushing. (M&N are the first to make a speculation like this.)
2003	Coop, M. R. Airey, D. W.	Carbonate sand	General Review	-Several Sands Referenced (Mostly Dogs Bay calcareous sand)	None	- Main focus with regard to monotonic shear testing is on critical state theory. States that the CS is not typically reached in triaxial testing because tests are usually terminated at 20 - is in fact straight and is not curved as frequently assumed. - Performs an excellent review of research on calcareous soils, with special attention on their basic properties. - Limited analysis of monotonic shear of uncemented calcareous sands

Table 2.1. A Brief Summary of The Studieis About the Shear Properties of the Calceraous Sands.

Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
2006	Sharma, S. S. Ismail, M. A.	Monotonic and cyclic behavior of two calcareous soil of two different origins	Offshore Platform (Monotonic & Cyclic)	-Goodwyn (GW) Fine-grained, offshore calcareous soil. - Ledge Point (LP) Coastal aeolian calcareous sand	CIU triaxial comp. and Cyclic CU triaxial	- Tested two calcareous soils. Showed that the soil with weaker soil particles, also at a greater void ratio, had significantly more excess pore pressure generation. - Higher strains are needed to reach the PT in calcareous sands. - The friction angle is greater, but high strains are required, more than 20
Year	Author(s)	Title	Motivation	Soil Type	Shear Tests	Significant Monotonic Shear Findings
2006	Cataño Arango, J.	Stress strain behavior and dynamic properties of Cabo Rojo calcareous sands	General study of Puerto Rican calcareous sand.	-Cabo Rojo, fine to medium calcareous poorly graded sand - South Bend, fine to medium silica poorly grades sand	Direct shear and CIU triaxial comp.	- Calcareous sand is more ductile, higher strains are needed to reach peak strength. - Calcareous sands are weaker, have a lower deviator stress at failure. - More compressive, greater development of positive excess pore pressure during shear
2010	Brandes, H. G.	Simple Shear Behavior of Calcareous and Quartz Sands	Maui Dune, Kawaihae Harbor and East Island	Holocene age and reflect a mineralogy dominated by aragonite and magnesiumrich calcite typical of very young sediments	Cyclic simple shear test	-Measured friction angles for the calcareous sands are not as high as reported in other studies because of significant particle fracturing and rounding caused by the high-energy, shallow-water environment. -Particle breakage and rounding during shearing can cause a substantial decrease in friction angle at higher normal stresses. This decrease is more severe in the calcareous sands due to lower grain hardness and more prevalent intraparticle voids. -Cyclic liquefaction resistance is higher for the calcareous sands than for the quartz sand. This is attributed to differences in grain geometryand gradation. -As expected, cyclic resistance increases with increasing density (or decreasing void ratio).

2.3. Fly Ash Pellets

Artificial lightweight aggregates which consist primarily of fly ash, fine and/or coarse aggregates and a small amount of Portland cement are being used increasingly in the construction industry. Pelletization is a promising process for making artificial aggregates from fine-grained materials like fly ash, which facilitates its high volume utilization as lightweight aggregate in concrete (Danyıldız, 2007).

2.3.1. Fly Ash

To understand the theory of pelletization with fly ash or cement and the behavior of pellets produced from fly ash, the material fly ash is needed to be understood. According to the American Concrete Institute (ACI) Committee 116R, fly ash is defined as the finely divided residue that results from the combustion of ground or powdered coal and that is transported by flue gasses from the combustion zone to the particle removal system (ACI Committee 232, 2004). Fly ash is a major byproduct of coal burning in thermal power plants. The disposal of such fly ash is creating a serious problem because of its storage space and cost involved in its storage. At the same time there is a lot of pollution of the environment due to the fineness of the fly ash. The effects for its utilization for many gainful purposes have been made since late sixties of this century by various research institutions and public enterprises. Some researchers have investigated methods to beneficiate high-carbon fly-ash into a marketable product. High percentage of fly ash is utilized in developed countries. The majority of the fly-ash produced every year is not being used; nearly 80% of the fly ash produced is still land filled (Danyıldız, 2007).

According to ASTM C-618 there are two different category for fly ash : Class F and Class C. The classification is mainly based on if the fly ash in question carries only pozzolanic (Class F) or pozzolanic and cementitious (Class C) property. The Class F fly ash is normally generated due to combustion of anthracite or bituminous coal whereas the Class C fly ash is produced due to burning of lignite or sub-bituminous coal. The main oxides forming fly ash are SiO_2 , Al_2O_3 , and Fe_2O_3 , and they also contain

significant amounts of CaO, MgO, MnO, TiO₂, Na₂O, K₂O, SO₃, etc. ASTM Class C fly ash (high lime fly ash) typically contains CaO in excess of 10 % up to 40 %, and Class F fly ash (low-lime fly ash) generally contains less than 10% CaO. In general, Class C fly ash has higher CaO + MgO + SO₃ and lower Al₂O₃ + SiO₂ compared to Class F fly ash. Due to high CaO content, Class C fly ash participates in both cementitious and pozzolanic reactions, whereas Class F fly ash participates only in pozzolanic reaction during the hydration process. Class C fly ash is classified as cementitious and pozzolanic admixtures/ additives and Class F fly ash is classified as normal pozzolan for use in concrete.



Figure 2.5. Full Scale Pelletizer (Baykal, 2015).

The quantity and quality of the fly ash produced are a function of several factors, among which the coal source and method of production are the most influential on the physical, chemical properties and engineering performance of the final product during utilization processes (Meyers *et al.*, 1976).

The origin of coal was noted to be the most influential factor on the properties of the fly ash, the factor by which the chemical properties are determined and so its

class. The degree of coal pulverization affects mainly the physical properties of fly ash, mainly the grain size distribution, the fineness, the specific surface area, the specific gravity, -and the loss on ignition (due carbon content) etc.

Fly ash consists of inorganic matter present in the coal that has been fused during coal combustion. This material is solidified while suspended in the exhaust gases and is collected from the exhaust gases by electrostatic precipitators. Fly ash is made up of fine, spherical particles with a diameter range of 0.074 to 0.005 mm. These hard and round particles are so small that in laboratory tests for fineness. Under a microscope, fly ash particles look like tiny ball bearings. A SEM of typical fly ash particles is given in Figure 2.2 (Danyıldız, 2007).

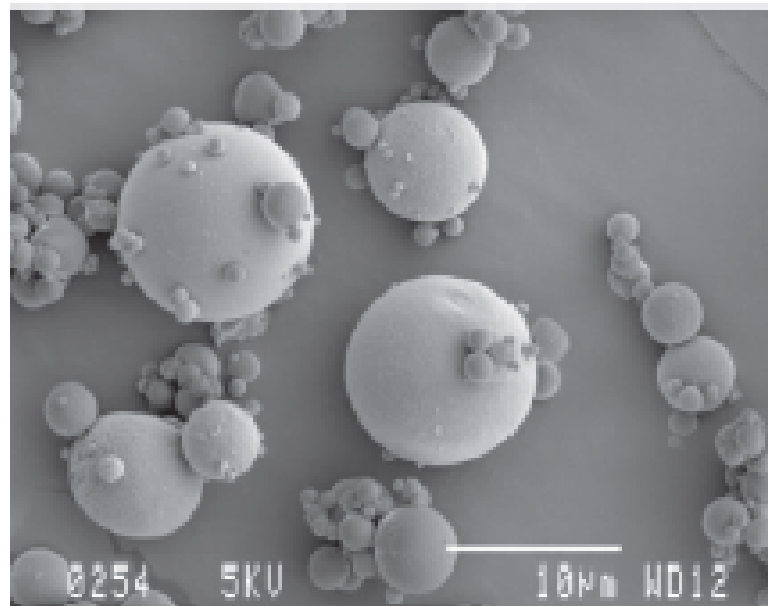


Figure 2.6. SEM of Typical Fly Ash Particles (Danyıldız, 2007).

Silica is the primary compound in fly ash. Fly ash is a pozzolan. A pozzolan is primarily a siliceous or siliceous and aluminous material, in a finely divided form and in the presence of water, chemically reacts with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (Danyıldız, 2007).

Formation of cementitious material by the reaction of free lime (CaO) with the pozzolans (AlO_3 , SiO_2 , Fe_2O_3) in the presence of water is known as hydration. For

class C fly ash, the calcium oxide (lime) of the fly ash can react with the siliceous and aluminous materials (pozzolans) of the fly ash itself. Since the lime content of class F fly ash is relatively low, addition of lime is necessary for hydration reaction with the pozzolans of the fly ash. For lime stabilization of soils, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil. The pozzolanic reactions are as follows:

- $Ca(OH)_2 \Rightarrow Ca^{++} + 2[OH]^-$
- $Ca^{++} + 2[OH]^- + SiO_2 \Rightarrow CSH (Silica\ gel)$
- $Ca^{++} + 2[OH]^- + Al_2O_3 \Rightarrow CAH (Alumina\ gel)$

Hydration of tricalcium aluminates in the ash provides one of the primary cementitious products in many ashes. The rapid rate at which hydration of the tricalcium aluminate occurs results in the rapid set of these materials, and is the reason why delays in compaction result in lower strengths of the stabilized materials. The hydration chemistry of fly ash is very complex in nature. So the stabilization application must be based on the physical properties of the ash treated stabilized soil and cannot be predicted based on the chemical composition of the fly ash (Danyıldız, 2007).

Fly ash can be used in various developmental and construction activities. Some of the prominent uses of fly ash are given as under:

- For development of various value added products, fly ash based bricks, blocks, mosaic tiles, glazed floor, wall tiles, partition boards, concrete products, cellular light weight concrete blocks etc.
- For reclamation of waste land/filling of low lying areas, soil amendment.
- As mine fills in open cast mines and the underground mines can be stowed with ash.
- Production of cement.
- For use in concrete as lightweight aggregate.
- In construction of embankments, dykes, road sub-base and base course.
- In agriculture as a source of micronutrient for improvement of soil condition.

- Road-building applications.

2.3.2. Properties of Fly Ash

The grain size distribution of the fly ash and the pulverized coal during the combustion process are different due to agglomeration of ash particles in molten stage. In general, the amount of fly ash retained (by weight) on the 80 μm sieve ranges from 6 to 25 %, on the 50 μm sieve from about 16 to 40 %, and on the 45 μm sieve from about 3 to 14 % (Berry, 1976).

2.3.3. Pelletization

Anderson was the first to put the idea of pelletization process. Brackbelsberg revised process with the addition of binder to the fines being pelletized and the pellets would be strengthened at elevated temperatures during production (Yukio *et al.*, 1999).

Two basic steps have to be performed to produce an aggregate from fly ash: bonding of the material in the pellets by firing, hydrothermal processing, and cold bonding which gives the pellets strength and other properties necessary to meet the criteria for a lightweight aggregate, and agglomeration of the fine fly ash particles to form aggregate size pellets (Bijen, 1986).

Agglomeration (also called pelletization) can be performed by two distinct methods. One is by agitation, where fly ash particles are introduced onto an inclined rotating disk along with a wetting agent and an appropriate binder as shown in the Figure 2.8. Balling of the material occurs by the formation of seeds which ultimately grow into pellets of a certain maximum size. The product obtained at the end of the process is termed as fresh pellet. The crushing strength of the fresh pellet must be adequate for hauling and stockpiling purposes (Döven, 1998).

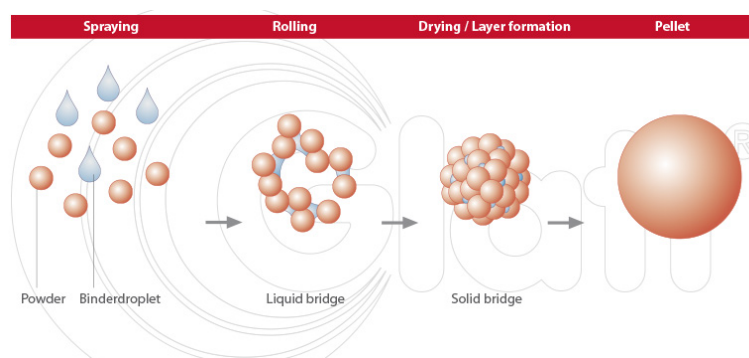


Figure 2.7. Formation of Pellet Particles (<http://www.glatt.com/ru/processy/pelletirovanie/>).

The other method is the pressure or extrusion method where agglomeration is accomplished by using a continuous piston-type press where more or less rectangular or cylindrical pellets are formed as shown basically in Figure 2.9. The extrusion process generally results in a product having a higher density than the spherical pellets produced by agitation (Bijen, 1986).

Regardless of which manufacturing process is used, pellets have to meet the following requirements (Vuppala *et al.*, 1997):

- They should be near spherical and have a smooth surface; both considered optimum characteristics for subsequent film coating.
- The particle size range should be as narrow as possible. The optimum size of pellets for pharmaceutical use is considered to be between 600 and 1000 μm .
- The pellets should contain as much as possible of the active ingredient to keep the size of the final dosage form within reasonable limits.

The pellets may also be sintered during a specified interval of time after production depending on concerns like properties of the fines used in pelletization process, the purpose of use and the rate of increment in engineering performance of pellets.

Besides that, several advantages of cold bonding process based on experiences to date show that:

- Cold bonded process plants tend to have lower capital costs than sintering/rotary kiln firing plants.
- Cold bonded processes tend to have lower operating costs than sintering/rotary kiln firing.
- Cold bonded process plants have less environmental impact than sintering/rotary kiln firing plants (no exhaust gases). Both processes require particulate emission controls and may have waste water streams.
- Finished material costs tend to be lower for cold bonded material than for sintered material (Danyıldız, 2007).

2.3.4. Manufacturing of Pellets with Fly Ash

Fly ash is a fine grained material which is the waste material of the combustion of pulverized coal in power station furnaces. As it is too fine, it is collected in mechanical or electrostatic separators. The residue extracted from the bottom of boilers is not considered as fly ash.

Fly ashes that give reaction with Ca(OH)_2 at room temperature can act as pozzolanic materials. The pozzolanic activity is attributable to the presence of SiO_2 and Al_2O_3 in amorphous form.

2.3.5. The Crushing Behavior of fly Ash Pellets

As fly ash pellets exhibit a crushing behavior and classified as crushed sand (Döven, 1998), the crushing strength parameters have been investigated for fly ash pellets by some researchers. Arslan studied particles at 5-5.5 mm diameter range, the results are found to range between 75 N/cm² to 96 N/cm² (Arslan, 2003). The results of the previously done crushing strength tests by Danyıldız (2007) are given in the table below.

Table 2.2. The Crushing Strength Values for Different Curing Time (Danyıldız, 2007).

Day	$F_{average}$ (N)	$d_{average}$ (cm)	$\sigma_{average}$ (N/cm ²)
7	15.18	1.27	11.83
14	65.55	1.29	49.88
28	246.1	1.24	201.62

2.4. Particle Shape Quantification and Image Processing

Investigating particle shape was a method of geology field prior to geotechnical engineering field. Particle shape is considered to be the result of two main factors. First is the difference in transportation agents bringing the rock from its original place to deposit and the second factor is the particle genesis itself (rock structure, mineralogy, hardness, etc.).

There are three different scales used to describe particle morphology, shape or form, angularity or roundness, surface texture or roughness. Form or shape factor deals with the gross form of a particle (axis lengths, perimeter, surface area, volume, etc.) where angularity or roundness reflects the number and sharpness of corners. According to Wadell form and roundness are mutually independent factors (1932). Moreover, surface texture or roughness provides details about the number, size and sharpness of the asperities along the particle surface and on the corners (Sukumaran and Ashmawy, 2001).

On the early stage of morphology measurements, standard two dimensional charts developed by Rittenhouse, Powers, Krumbein and Krumbein and Sloss were in use to compare each particle to the standardized shapes in the chart.

With progression in digital image capturing and processing, variety of mathematical definitions were suggested by different authors for sphericity (2D form) and

roundness over years, which finally resulted in definitions suggested as:

$$\text{For roundness: } R = \frac{4A}{\pi L_{Major}^2} \quad (2.2)$$

$$\text{For sphericity: } S = \frac{4\pi A}{p^2} \quad (2.3)$$

where P is the perimeter of the particle, A is the cross-sectional area, and L_{Major} is the length of the major axis.

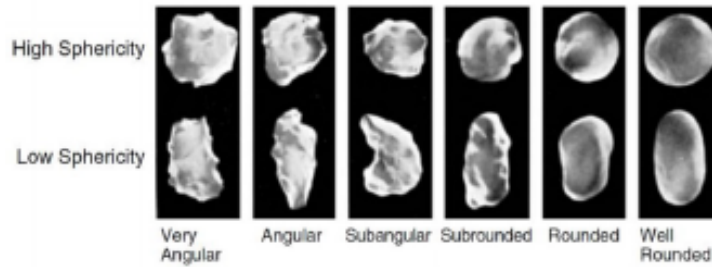


Figure 2.8. Qualitative Sphericity-Roundness Chart (Powers, 1953).

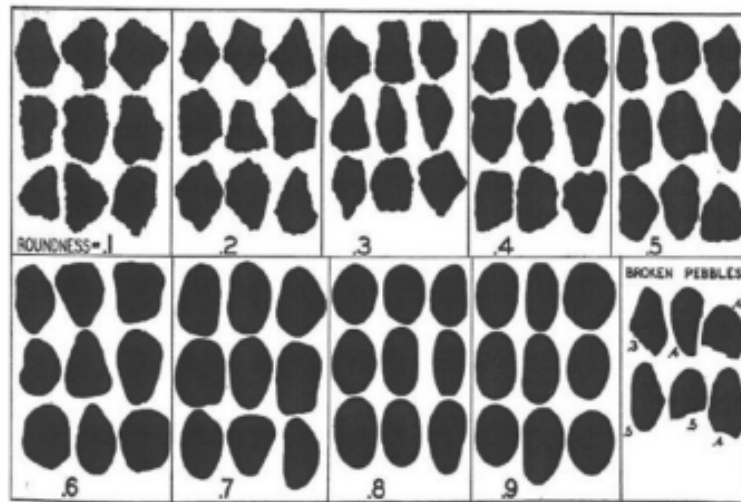


Figure 2.9. Comparison Chart for Roundness (Krumbein, 1941).

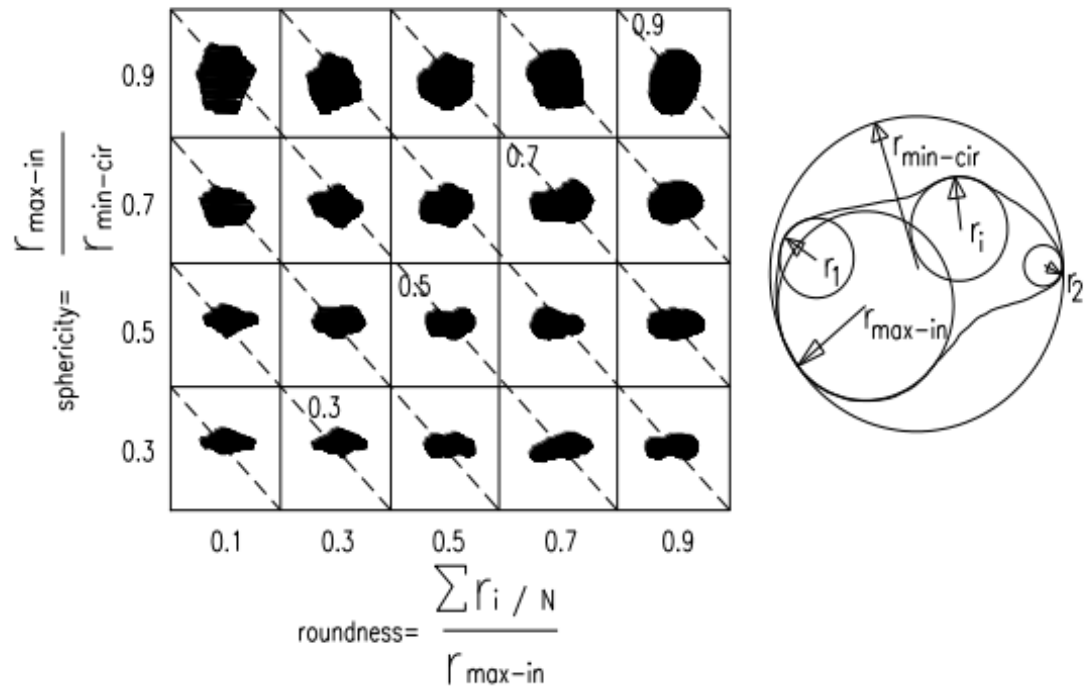


Figure 2.10. Particle Roundness- Sphericity Chart (Krumbein and Sloss, 1963).

The image processing program used in this study was ImageJ . It is a Java based open sourced image-processing tool adapted from an earlier Macintosh version of the National Institute of Health (NIH) Image program. This program is available over the public domain and is able to assess numerous image formats with numerous built-in editing and analyzing tools. This program has many advantages such as high speed, easy conversion of picture to binary format, attribution of a specific number to each processed particle and easy modification of image's brightness/contrast, threshold, and color balance.

According to thorough investigation performed by Cox (2008), one of the most accurate ImageJ plugins is Shape Descriptors. This plugin uses the formulae given above to calculate sphericity and roundness.

3. METHODOLOGY

The main objective of performing this research is to understand how the cyclic behavior of fly ash pellets differs from same dimensioned sands. Most liquefaction research has been performed on quartzitic sands (Kramer, 1996). It is not clear if the current liquefaction evaluation procedures, which are largely based on case histories from sites with quartzitic sands, are directly applicable to same sized pellets. This study investigates how the particle rearrangement and crushing mechanism in sand and fly ash pellets are manifested during cyclic loading. The specific objectives of performing this research are to:

- (i) Evaluate the undrained static stress-strain behavior of fly ash pellets using monotonic triaxial testing. Compare this data to the data of the same graded sand samples. This provides a preliminary insight for the cyclic loading.
- (ii) Evaluating the cyclic behavior of fly ash pellets. Comparing these results with trends of Sile and Kilyos sands at similar relative densities and loading conditions provide a good idea for understanding the properties of newly produced material.

3.1. Materials

In this study, two different kinds of sand and two different sized fly ash pellets are used. Their properties will be explained in this chapter.

3.1.1. Natural Sands

Two different kinds of natural sand, Sile and Kilyos sands, are used to be able to give a good perspective about the behavior of fly ash pellets. Their index parameters are already investigated by other researchers in Karl Terzaghi Soil Mechanics Lab since these samples are frequently used in laboratory works. Their gradation characteristics can be seen in Figure 3.1 and Figure 3.2.

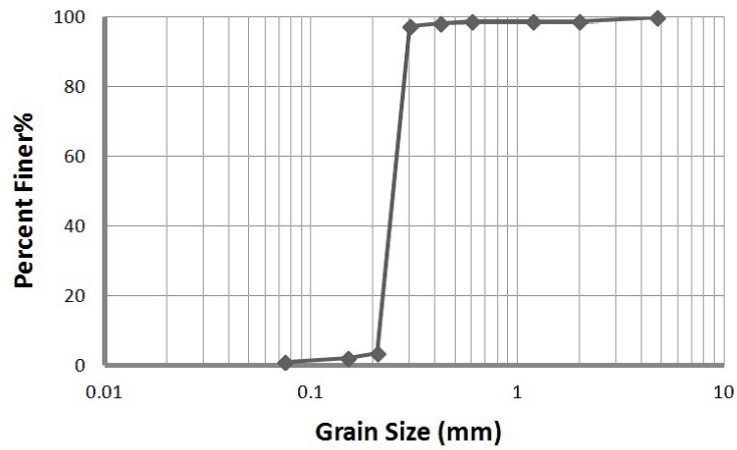


Figure 3.1. Kilyos Sand Size Distribution (Ranjpour, 2015).

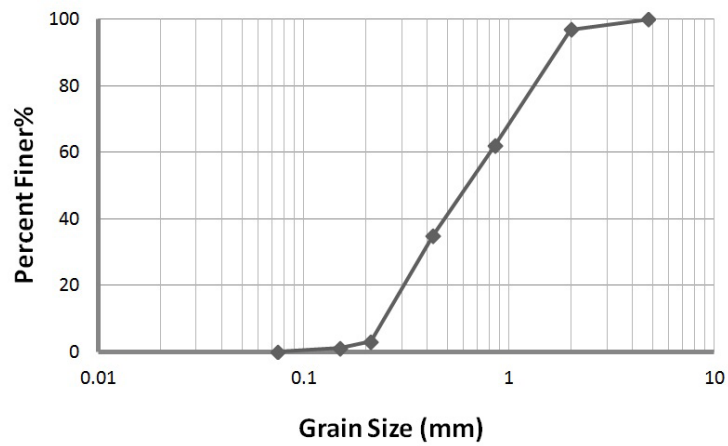


Figure 3.2. Sile Sand Size Distribution (Ranjpour, 2015).

The index properties of the sands are given in Table 3.1. ASTM D854-10 standard tests are conducted for obtaining the specific gravity.

Table 3.1. Properties of Sile and Kilyos Sands.

Sand Type	Specific Gravity	Maximum Void Ratio	Minimum Void Ratio	D ₅₀ (mm)
Sile	2.61	0.78	0.52	0.26
Kilyos	2.66	0.77	0.44	0.65

3.1.2. Fly Ash Pellets

Pelletization is simply a method for producing bigger particles by using smaller particles by using the pelletization theory.

Artificial lightweight aggregates which consist primarily of fly ash, fine and/or coarse aggregates and a small amount of Portland cement are being used increasingly in the construction industry. Pelletization is a promising process for making artificial aggregates from fine-grained materials like fly ash, which facilitates its high volume utilization as lightweight aggregate in concrete (Danyıldız, 2007).

The ability to produce sand or gravel particles at the exactly needed properties by using fly ash is the most interesting part of implementation of this method in geotechnical engineering. Moreover this production can be integrated directly to the facilities that produce fly ash, easily and with few costs.

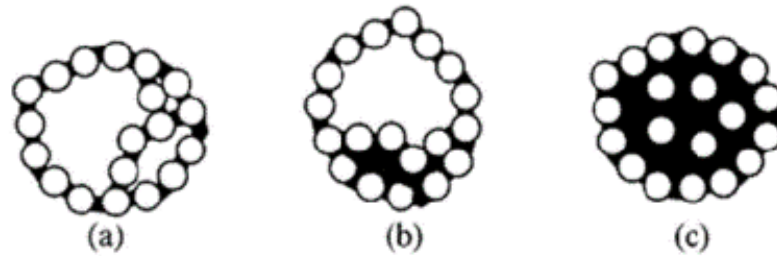


Figure 3.3. Mechanism of Pellet Formation (Jaroslav and Ruzickova, 1988).

The index properties of fly ash pellets that are used in this study are investigated. The tests are done to obtain the index properties such as minimum void ratio, maximum void ratio and unit weight. These findings are given in Table 3.2.

3.1.2.1. Pelletization Theory. The pelletization theory was developed in 1940's. The studies showed that the performance of pelletization process is a function of:

- (i) The engineering properties of the material pelletized;

Table 3.2. Maximum and Minimum Void Ratios.

Fly Ash Pellet Type	Maximum Void Ratio	Minimum Void Ratio	D ₅₀ (mm)
Well graded	1.08	0.57	0.26
Poorly graded	1.13	0.66	0.65

- (ii) The amount of moisture in the medium;
- (iii) The mechanical process parameters such as the angle of balling drum or disc to the normal and the revolution speed.

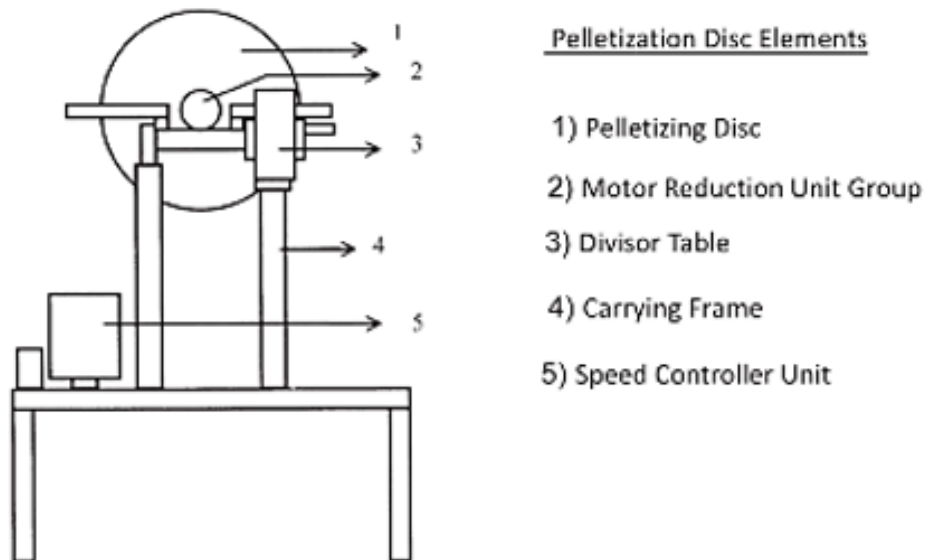


Figure 3.4. Sketch of Pelletization Disc Unit (back view) (Döven, 1998).

Observations and analysis performed on these parameters with respect to mechanic and kinetic laws formed the theory of pelletization process. When a fine-grained material is moisturized, there forms a thin liquid film on the surface of the grains, which forms meniscus between the grains, structures like bridges. In case the particles are rotated in a falling drum or disc, then they form ball shape structures with enhanced bonding forces between grains due centrifugal and gravitational forces as shown in Figure 3.3 (Jaroslav and Ruzickova, 1988).

3.1.2.2. Fly Ash. The fly ash is supplied from Soma Thermal Power Plant, located in the Aegean region of Turkey. Locally produced low calorie lignite coal is used in this power plant. Approximately 30 per cent by weight of this coal turns into ash upon burning. Soma fly ash is self-cementitious, and four million tons are produced annually. The fly ash is mixed with water and disposed into an ash dam. Upon collection directly from the hopper at the power plant, the fly ash was sealed in bags and transported to the Karl Terzaghi Soil Mechanics and Geotechnical Engineering laboratory. Its exposure to air, moisture and light is prevented. The physical and chemical properties of fly ash was determined in AkçanSa cement factory laboratories in Büyükçekmece, Istanbul and is presented in Table 3.3 (Saygili, 2008).

Table 3.3. Physical and Chemical Characteristics of Fly Ash (Saygili, 2008).

Analysis Report												
Chemical analysis	SiO ₂ (%)	Al ₂ O ₃ (%)	Fe ₂ O ₃ (%)	CaO (%)	MgO (%)	SO ₃ (%)	Na ₂ O (%)	K ₂ O (%)	CI (%)	Loss on Ignition (%)	Specific Weight	Specific Surface Area (cm ² /g)
Fly Ash (FA)	36.94	17.2	4.76	33.22	1.36	3.82	0.34	1.82	0.0045	0.19	2.56	3206
Physical analysis		Fineness (%)	45 mm	90 mm	200 mm							
Fly Ash (FA)			23.3	9.93	2.74							

3.1.2.3. Pelletization Process. According to previous studies the regression analysis results indicated that for 43° tilt angle and 45 rpm, the highest values for unit weight and crushing value were obtained. The formation of pellets occurred between 6 and 9 min. The total pelletization time was determined to be 20 min, since it was found that under the mechanical and capillary forces, the saturation point in volume compression was reached around this time.

Trial pellet production was done by observing of pellet formation stages, the shape and apparent strength of pellets with changing the angle and the revolution speed of the pelletizing disc. During the optimization, firstly, the angle and revolution speed parameters were tried to be determined for the desired pelletization process. A simple method was used for the determination of apparent strength of fresh pellets

to investigate if they have adequate strength for hauling and stockpiling conditions in which the pellets are let to fall from a 115 cm height (Danyıldız, 2007). The result of optimization study was in accordance with Döven (1998). The execution of theoretical equations with defined parameters and observations on fresh pellet formation and strength showed that operation angle should change between 35° and 50° and revolution speed between 35 and 55 rpm (Döven, 1998). It is found that 45° rpm and an operation angle of 45° are adequate for pellet formation both for fly ash. During the production process of pellets, it is observed that the water droplets are smaller with a tight nozzle of water spray and it causes production of pellets with smaller diameter. The formation of pellets occurred between 6 and 9 minutes. The total pelletization time was about 20 minutes, since it was found that under the mechanical and capillary forces, the saturation point in volume compression was reached around this time. Avoiding water film on the surface of the pellets was of extreme care during production. The pellets were kept in plastic bags in the curing room in which the temperature and relative humidity were 21°C and 70% respectively. Cement or fly ash was mixed with water in pelletization disc and the samples were put in curing room, in order to get the needed strength before they were used in experiments (Döven, 1998).

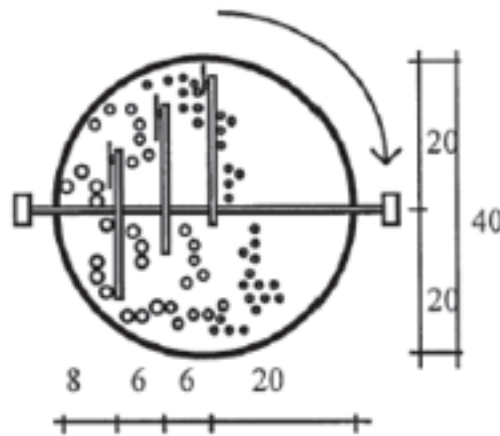


Figure 3.5. Sketch of Scrapping Pellets Positioning (Erdurak,2011).

During this production two main particle sizes are obtained first group is between 8mm to 16 mm and second group is between 2mm to 4mm. For the first group blades positioned at 10, 15 and 25 cm of the disc, formation of pellets were occurred between

3 min and 6min, water is added in larger drops. For the second group blades positioned at 5, 10 and 15 cm of the disc, formation of pellets were occurred between 6 min and 10 min, water is sprayed finer. Around 250 kilograms of pelletized fly ash was produced with approximately 75 % efficiency for the specified sieve intervals. Water content is between 20-25 % for both gravel size groups and no additives were used during production, the pellet formation is just fly ash and water. The pellets are formed and sieved to obtain the desired size and gradation for this study. The change in the grain size of fly ash can be seen in the Figure 3.6.



Figure 3.6. The Grain Size Evolution of fly Ash Pellets.

3.2. Experimental Setups and Procedure

In this chapter, previously the materials and the manufacturing techniques are explained, then the experiments are described.

3.2.1. The Shape Factor Analysis

The image processing program used in this study is ImageJ (Rasband, W. S., 2004). It is a Java based open sourced image-processing tool adapted from an earlier Macintosh version of the National Institute of Health (NIH) Image program. Pictures were taken using an Mshot brand, MD50 model digital light microscope camera. Care was taken to prevent particles to overlap while capturing the images, so the processing would be easier and more precise (Figure 3.7). Then the captured images were processed using ImageJ software. Then using ImageJ unclear particles were discarded from the images and the rest were taken into account for shape factor analysis.

The shape descriptor plugin was used in this study to obtain sphericity and roundness values. Cox used a simple circle to investigate differences in the shape parameters from the two plugins of ImageJ as compared to the theoretical values associated with a circle. Both manually sketched circle and “Circle Test” plug in, which automatically draws a circle of radius 60 pixels in an image of size 324 pixels x 200 pixels, to investigate the outcome values was used. The discrepancy between the plug ins and the theoretical value in sphericity is more than roundness. A difference of about 10% lower is observed in the plug ins values as compared to the theoretical.

Table 3.4. Comparison of Theoretical and Predicted Shape Parameter Values.

Grain Shape Parameter	Manual	Plugin	Theoretical
Sphericity	0.908	0.91	1.000
Roundness	1.000	1.000	1.000

Each image was turned into binary as shown in Figure 3.8 since ImageJ is only capable of processing this type of picture.

The program ascribes a label to each particle, and any other dark stain in the picture

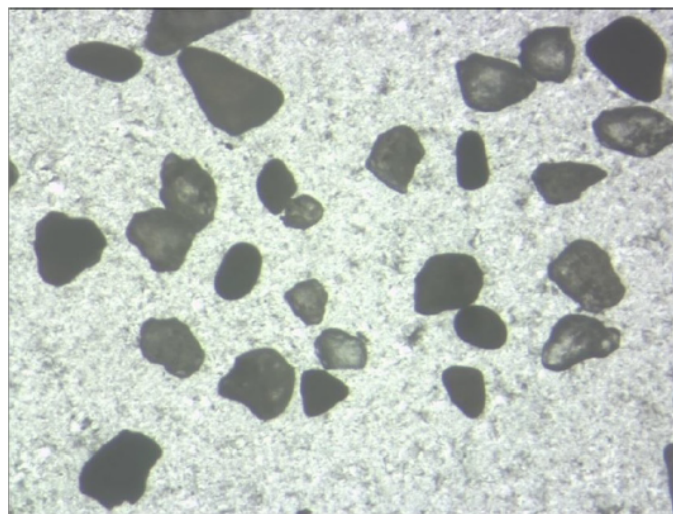


Figure 3.7. Example Image of Some Sand Particles.



Figure 3.8. Example Binary Image.

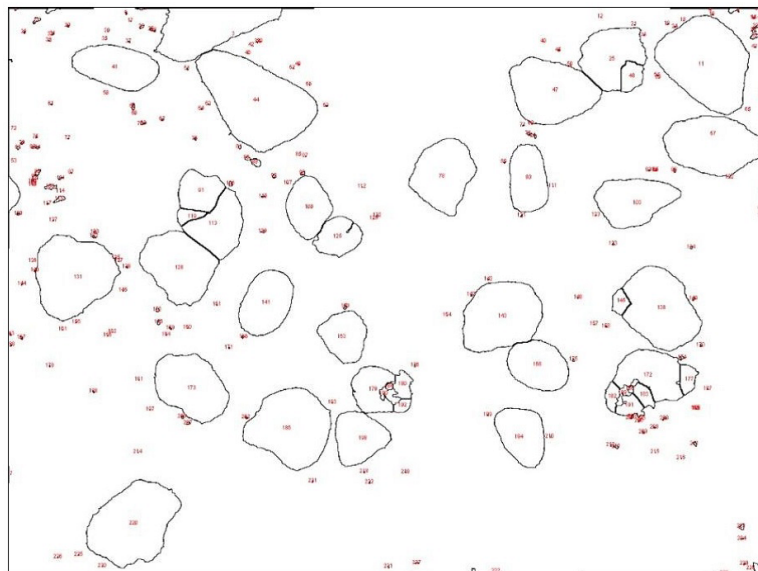


Figure 3.9. Example Image of Numbers Ascribed to Sand Particles.

Single particle values of 1-1							
File	Edit	Font					
Slice	Particle	Area	Perimeter	Angle	Form Factor	Round	Comp
1	1	1506.00	393.34	79.59	0.12	0.60	0.78
1	2	385.00	134.08	14.24	0.27	0.61	0.78
1	3	11567.00	1148.09	0.89	0.11	0.29	0.54
1	4	3496.00	839.40	176.98	0.06	0.15	0.39
1	5	9534.00	506.15	5.77	0.47	0.46	0.68
1	6	6645.00	475.32	91.91	0.37	0.74	0.86
1	7	4.00	7.66	90.00	0.86	0.68	0.83
1	8	24052.00	629.70	39.35	0.76	0.78	0.88
1	9	69.00	38.04	91.69	0.60	0.44	0.67
1	10	8.00	10.49	162.70	0.91	0.70	0.83

Figure 3.10. Example ImageJ Output Table.

Since the program calculates the value of each shape parameter for every particle, clear or unclear, the clear particles were chosen manually. Then the sphericity and roundness values for chosen particles from all the captured pictures were gathered in an excel file.

Table 3.5. Number of Analyzed Particles (Cox,2008).

Number of Particles	Mean C	Mean R	C STDEV	RSTDEV
15	0.716	0.708	0.053	0.137
25	0.716	0.723	0.040	0.1162
50	0.716	0.728	0.042	0.110
75	0.710	0.724	0.051	0.124
100	0.716	0.719	0.049	0.119
125	0.717	0.717	0.048	0.115
150	0.720	0.717	0.048	0.116
175	0.722	0.717	0.048	0.112
200	0.721	0.718	0.049	0.109
225	0.724	0.726	0.049	0.112
250	0.723	0.726	0.052	0.115
275	0.722	0.724	0.051	0.116
300	0.723	0.724	0.052	0.119
325	0.720	0.720	0.063	0.122
350	0.722	0.723	0.062	0.121

Processing images of 350 individual particles showed that using the average sphericity and roundness values of 50 particles would correspond to the minimum Standard Deviation for both parameters. So 50 was the number of particle processed for each sand type.

3.2.2. Crushing Strength Test

As in this study; the fly ash pellets, which behave as crushable sand, are studied, the crushing properties of the material has to be investigated. Single particle crushing test have been conducted by several researchers; such as Lee, Kwag et al. and Kato et al. For the practical determination of agglomerate strength, the tests frequently used are compression, drop and abrasion tests. The most recent tests were developed for the determination of impact, crushing value and shear strength as well as the resistance for bending and cutting (Pietsch,2001). The data defining “strength” obtained from these tests cannot be identified by theoretical studies because stress distribution does not have any uniformity and the strain leading to failure is normally not known (BS812, Part 110, 2010). In Figure 3.10 is given the sketch of crushing strength test apparatus.

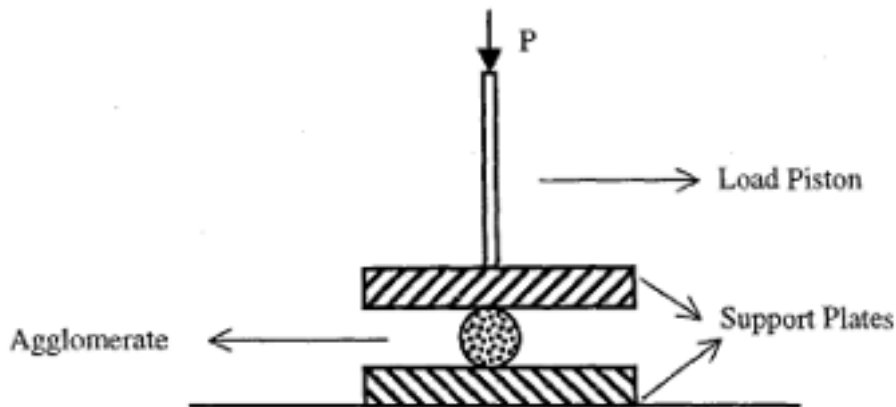


Figure 3.11. The Crushing Strength Test (Döven, 1998).

The crushing strength test is a simple test in which the individual pellets are placed one by one between two parallel plates and loaded with constantly increasing force P till failure occurs. The “crushing strength” or more often called as “crushing value” is defined as the mean statistical force applied at for failure for a number of agglomerates statistically representative. Normally, by dividing the force at failure to the projected area would give the “agglomerate crushing strength”, however this may not be acceptable since the stress distribution varies greatly depending on the shape of agglomerate. The test results are rarely comparable as in the case of perfectly rounded spherical pellets for which the stress distribution is uniform throughout any section.

Flat surfaces of pellets having less sphericity cause uncontrolled stress concentrations due frictional forces created between plate and pellet, which leads to the blocking of transverse expansion under loading (Sabry, 1977).

The crushing value test is performed with respect to the methodology given above and the results are expressed in terms of load in kilograms. The test was performed on each group of fly ash pellets as poorly and well graded. It is known that the results obtained for spherical pellets by this test give similar results to that obtained by point load test. Also the crushing value test is more practical and easier in execution with respect to point load test since the non-axial loading due shape irregularities is avoided (Sabry, 1977).

3.2.3. Cyclic Direct Simple Shear Test

Direct simple shear test (DSS) sample can be cylindrical or square samples, typically 50.8 mm to 63.5 mm in diameter with an approximate height of 25.4 mm. It is possible to confine the sample by a wire reinforced membrane or stacked rings. Consolidation phase is done under a constant vertical stress and followed by a shear phase. The values of the secant shear modulus can be calculated. The values of the secant shear modulus can be used to estimate the initial settlements of embankments built on saturated cohesive soils due to undrained shear deformations. The shear strength of a specimen depends on the method of shearing, soil type, normal consolidation stress, time of consolidation, rate of strain, and prior stress history of the soil (ASTM, D6528 - 07).

Lodd (1990) in his Terzaghi lecture clearly illustrated how to use the strength measured in direct simple shear to evaluate the stability of embankments on normally and over-consolidated clay foundation for initial conditions. He also showed how to incorporate the increase in undrained strength that occurs with consolidation by using the relationship among undrained strength measured in direct simple shear, vertical effective stress and pre-consolidation stress.

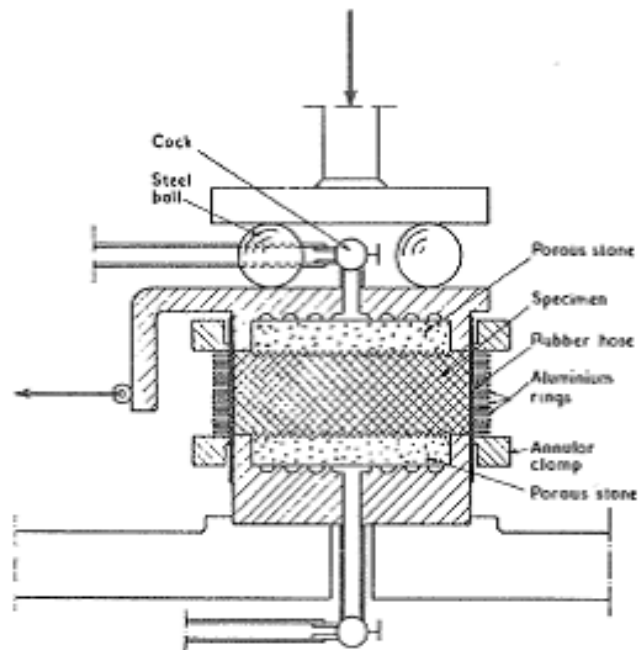


Figure 3.12. SGI Simple Shear Device 1936 (Kjellam, 1951).

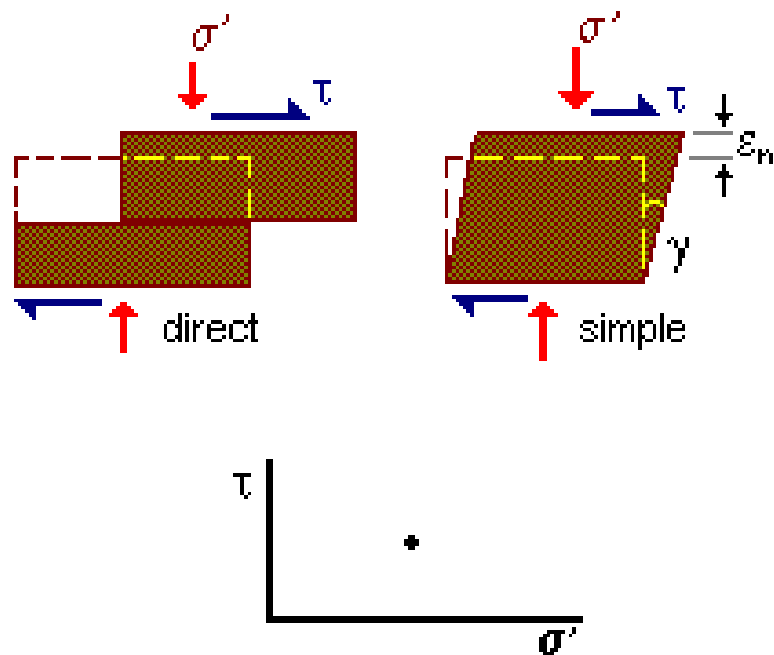


Figure 3.13. Direct and Simple Shear Comparison.

An example of one of these newer automated direct simple shear devices is a system created by the GEOTEC Corporation called the “Universal Shear Device”. This device is able to run monotonic and cyclic DSS tests under undrained conditions. The automated system dramatically reduces the labor involved in testing as the consolida-

tion steps and shearing phases are pre-set before the test is started. The breakthrough of this device comes from using PID adaptive control technology to apply vertical and horizontal forces to the test specimen.

The adoptive control permits rapid and precise control to produce the following operation:

- Application of constant vertical stress during consolidation;
- Maintenance of undrained conditions during shear by automatically adjusting the vertical stress to maintain constant specimen height;
- Application of a constant horizontal displacement rate during shear.
- Automatic acquisition of load, displacement, and pore pressure data.

With this system the steps to measure undrained shear strength for a stability analysis are reduced to:

- Prepare the sample to standard shape and size
- Mount the soil sample into membrane (surrounded by stack rings)
- Define the vertical stress for consolidation and shearing rate for shearing phase
- Run the test (after calibration the LVDT)
- Monitor progress of consolidation and shearing phases
- Dismount the sample and put it in oven-dry (to find moisture content)
- Prepare test report(Marr, 2003)

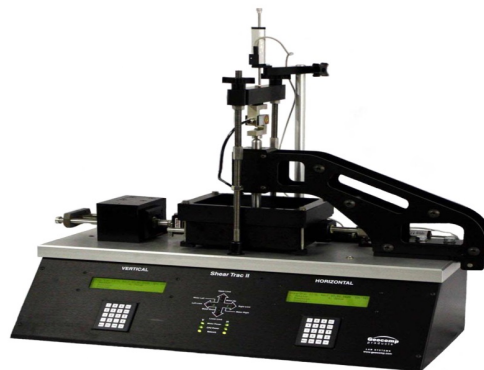


Figure 3.14. GEOTEC Direct Simple Shear Test Device.

3.2.4. Summary of ASTM Testing Requirements for Direct Simple Testing

According to ASTM D6528 the standard method for Consolidated Undrained Direct Simple Shear Testing Soils involves the following:

- (i) A specimen of cohesive soil is constrained axially between two parallel, rigid platens and laterally, such that the cross sectional area remains constant.
- (ii) The specimen is loaded axially and allowed to consolidate one dimensionally. Each normal load increment is maintained until excess pore water pressures are essentially dissipated as interpreted from interpretation of the axial displacement rate. The maximum normal load is maintained until completion of one cycle of secondary compression or one day longer than the end of excess pore water pressure dissipation.
- (iii) The specimen is sheared by displacing one platen tangentially relative to the other at a constant rate of displacement and measuring the resulting shear force. The platens are constrained against rotation and axial movement throughout shear.
- (iv) The specimen volume is held constant during shear to simulate undrained conditions. Constant volume is achieved by changing the normal load applied to the specimen to maintain constant specimen height. Since the pore pressure is zero through shear, the change in normal stress is equal to the change in effective stress and assumed to be equal to the change in pore water pressure that would occur in a sealed specimen confined by a constant total stress.

Figure 3.15 below provides a diagram outlining specimen set-up.

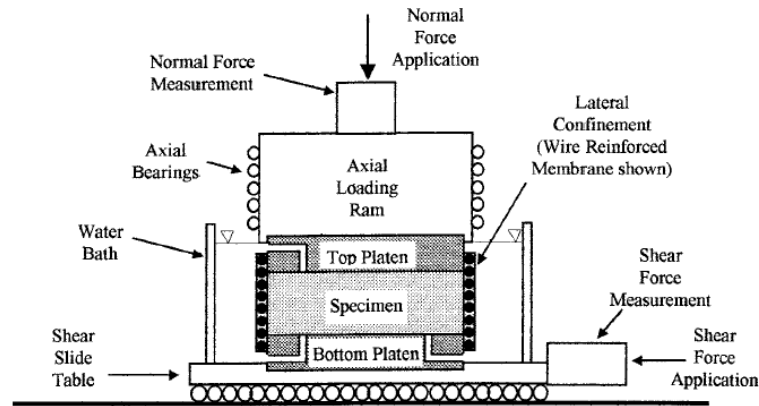


Figure 3.15. Schematic Diagram of Essential Direct Simple Shear Components (ASTM D6528).

3.2.5. Constant Volume / Constant Load Tests

Two main procedures are in use for the shear phase of direct simple shear tests: maintaining a constant vertical load during shearing or maintaining the volume of the soil specimen constant by keeping the height of the sample constant. The former is considered to yield drained shear strength parameters; the latter undrained shear strength parameters. For granular soils (sand), Direct Simple Shear tests are usually performed time drained with a constant load since the drained parameters are most likely to be investigated. For soft soils (clay, peat), the parameters of interest are more likely to be investigated in undrained conditions. In most of the simple shear apparatus, it is not possible to perform truly undrained tests since no back pressure can be applied with mean of cell pressure around the sample. So, the undrained simple shear response is normally investigated by performing drained tests at constant volume. Bjerrum and Landva (1966) proposed this equivalence between the two tests and furthermore, stated that the change in vertical stress observed in a constant volume test is similar to the change in pore pressure that would have occurred in a constant load undrained test.

Dyvik et al. (1987) performed tests to validate the “constant volume” hypothesis. They designed a chamber for the NGI Direct Simple Shear device that provides the capability of performing truly undrained tests with pore pressure measurements inside

the soil specimen. The authors compared truly undrained tests and constant volume drained test results on normally consolidated clay and obtained very similar results in both case. These conclusions only apply to normally consolidated saturated clay for shear strain lower than 10%. Unfortunately, it was not investigated for other materials. It needs not be correct for unsaturated soils. Budhu and Britto. (1987) performed a finite element modelling of the evolution of pore pressure in the core of the NGI specimen. Using Speswhite Kaolin properties and the modified Cam Clay model, they obtained results in accordance with Bjerrum's predictions for the same range of shear strain.

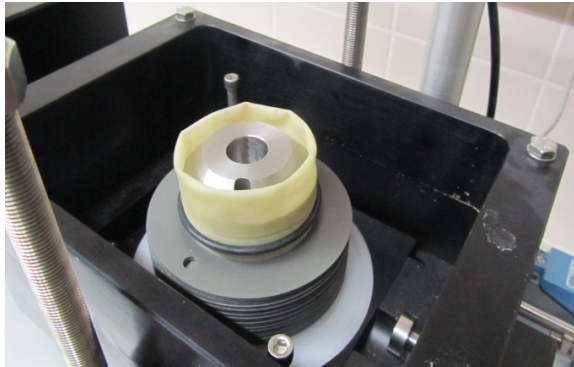


Figure 3.16. Samples Prepared for DSS Testing Using Stacked Rings.

Although stacked rings were developed over 50 years ago, wire reinforced membranes have been used almost exclusively in practice. Over the past 10 years automated testing systems such as the Geocomp Sheartrac-II have become readily available in the United States. These systems are tailor-made for DSS testing. Most of the manufacturers of these systems sell stacked rings as DSS test confinement types. Geocomp, Corp., GeoTac, and GDS Instruments are examples of companies selling stacked rings with automated shear systems. This has caused a significant increase in the popularity of stacked rings in the US. They have become the dominant confining system.

Use of the two confining systems may produce slightly different results in terms of measured stiffness, strength, and stress-strain behavior. This may be due to differing rigidity of the two systems, or some other mechanism (Baxter et al 2010). Below is a comparison the results of using stacked rings and wire reinforced membranes as a

confining system for direct simple shear tests:

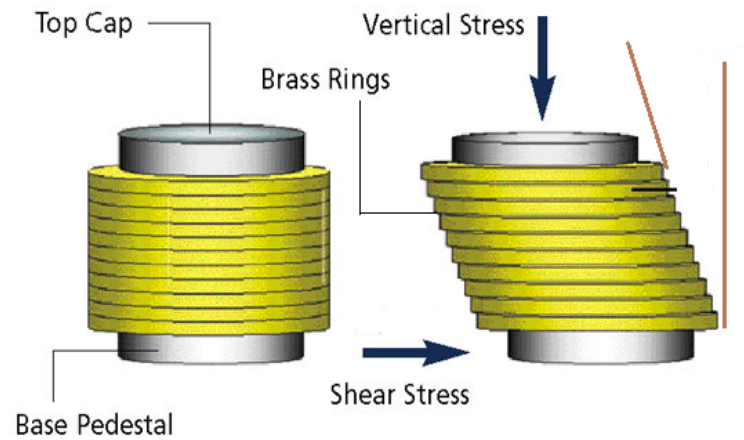


Figure 3.17. The Effects of Different Confining Systems (GEOCOMP DSS test manual 2012).

The initial stress states are the same as in direct shear test. With application of shear stresses, the lateral sides of the soil sample rotate through an angle with the initial vertical position as seen in Figure 3.18. Direct simple shear test apparatus are usually designed for drained and undrained condition during shearing, and monotonic and cyclic shear loading. The direct simple shear test has the advantage of applying a reasonable homogeneous shear stress state in the soil specimen and avoiding the stress concentrations as occurs in direct shear test.

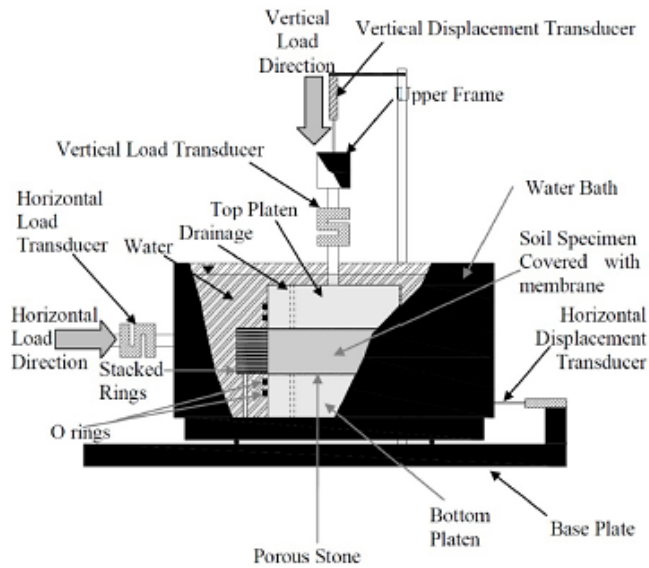


Figure 3.18. GeoComp Direct Simple Shear Apparatus (GEOCOMP DSS test manual 2012).

Cyclic simple shear testing is considered to be one of the most appropriate ways of reproducing in the laboratory the stresses that would be experienced by an element of soil in level ground subjected to earthquake loading. In the last few years cyclic simple shear testing has been analyzed experimentally and theoretically, and the advantages and limitations of the test presented. The main drawback concerns the lack of complementary shear stresses on the specimen vertical boundary and thereby a selection of the proper specimen size, that is, proper diameter versus height ratio must be carefully considered. The non-uniformity of the strain and stresses in the specimen most likely increases as the specimen diameter versus height ratio decreases.

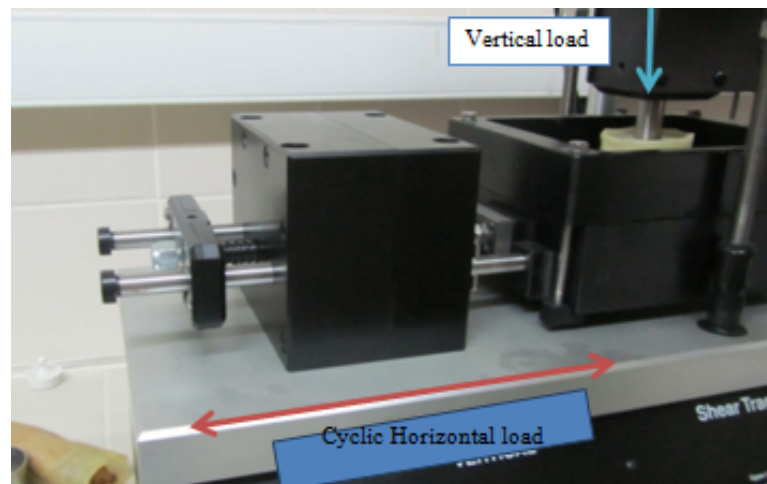


Figure 3.19. The Mechanism that Apply the Loads on the Cyclic Direct Simple Shear Test apparatus.

Geotechnical engineering problems associated with the dynamic loading of soils (cyclic loading) are many: wind or wave action, traffic activity, blasting, pile driving, traffic vibrations, machine vibrations, earthquakes, and so forth. They involve a wide range of strain amplitudes that may vary from very small, as in the case of some vibratory machine foundations, up to large amplitudes accompanying strong motion earthquakes and nuclear explosions. A solution to such problems requires a better understanding and knowledge of the dynamic (cyclic) soil properties and characteristics. Much research has been accomplished in the past few years on the study of cyclic soil properties using various laboratory and field measurement techniques. The object of many of these studies has been to evaluate the cyclic stress-strain properties and liquefaction potential of soil deposits under earthquake loading conditions. Cyclic soil properties are currently being evaluated using resonant column, cyclic triaxial, cyclic simple shear, torsional simple shear, and shaking table tests.

3.2.6. Experiment Procedure

In this study the CSS test were conducted in two phases:

- (i) Consolidation Phase

All specimens were consolidated prior to shearing. Testing reconstituted dry samples, the ultimate consolidation stress was chosen as 50 kPa and 100 kPa for 15 minutes. Monitoring cyclic tests it was observed that this time is enough to decrease the vertical displacement ratio to zero.

(ii) Cyclic Shearing

4 cyclic loading combinations were used for each material.

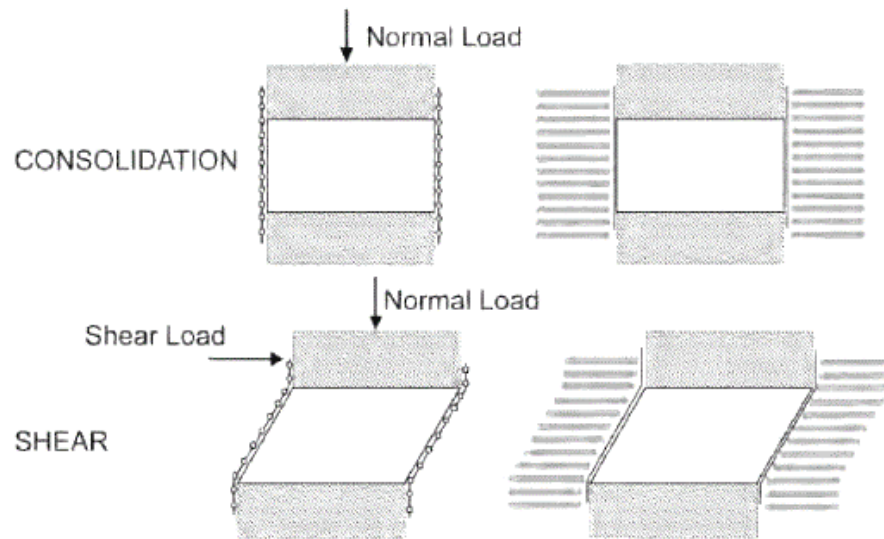


Figure 3.20. Direct Simple Shear Confinement Types: wire reinforced membranes (left) and stacked rings (right) (Baxter *et al.*, 2010).

Table 3.6. Cyclic DSS Tests CSR and CP Combinations.

Combination	CSR	CP
1	0.2	1
2	0.3	1
3	0.2	2

where CP is the cyclic period and CSR is the cyclic stress ratio. And CSR is defined for the testing apparatus as:

$$CSR = \frac{\text{Applied Shear Stress}}{\text{Applied vertical Stress}} \quad (3.1)$$

The ideal sinusoidal waveform may not be achieved easily. The waveforms are acceptable provided that the difference between the amplitudes and durations of successive half-cycles is less than 10%, the noise (ringing) in the waveform is not significant, and there are not any prominent spikes near the peaks of waveform (Silver, 1977). The unacceptable waveforms can cause abnormal changes in pore water pressure resulting in significant deviations from the actual soil response to an ideal sinusoidal waveform.

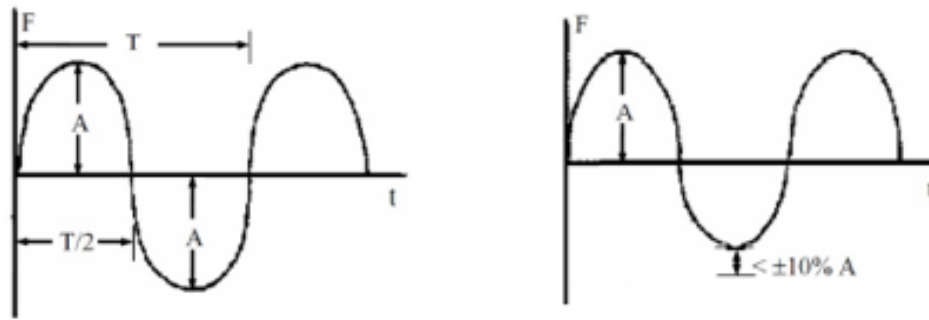


Figure 3.21. An Ideal Waveform and the Limit Value of 10% Difference for Two Successive Half-Cycles (Silver,1977).

The ShearTrac II apparatus uses a closed-loop load control system in order to provide a load-history consistent with that of an ideal sinusoidal waveform. It uses a parameter called “Update A” for automatic updating of the load increments in a closed-loop loading system during a cyclic loading test due to nonlinear response of the material. Update A factor is dependent on several factors such as the stiffness and the dimensions of soil specimen, frequency of loading (f), and calibration factors of the apparatus, and can be obtained using a calculation tool provided by the manufacturer. Other important parameter in the algorithm of closed-loop control, P-Gain, I-Gain and D-Gain were kept at default values as it was suggested in the apparatus instruction manual.

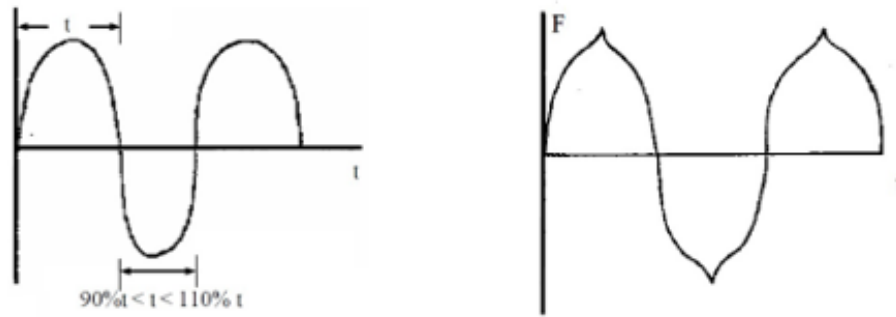


Figure 3.22. The Limit of 10% Difference in Duration of Successive Half-Cycles and the Unacceptable Waveform Due to Spikes at Peaks (Silver, 1977).

3.3. Evaluation of Data

The cyclic simple shear test is done as it is explained before by using the ShearTrac II apparatus. A series of cyclic simple shear tests were conducted on each sand type with different cyclic stress ratio (CSR) and Cyclic Period (CP) combinations in order to investigate the cyclic behavior of each material under each condition.

The specimens were consolidated under 50 kPa and 100 kPa vertical stresses and then sheared with cyclic simple shear loading until failure due to liquefaction.

The values given by the cyclic direct simple shear test apparatus can be seen in the Figure 3.23 and Figure 3.24. As it is seen, the values are given as vertical displacement in time.

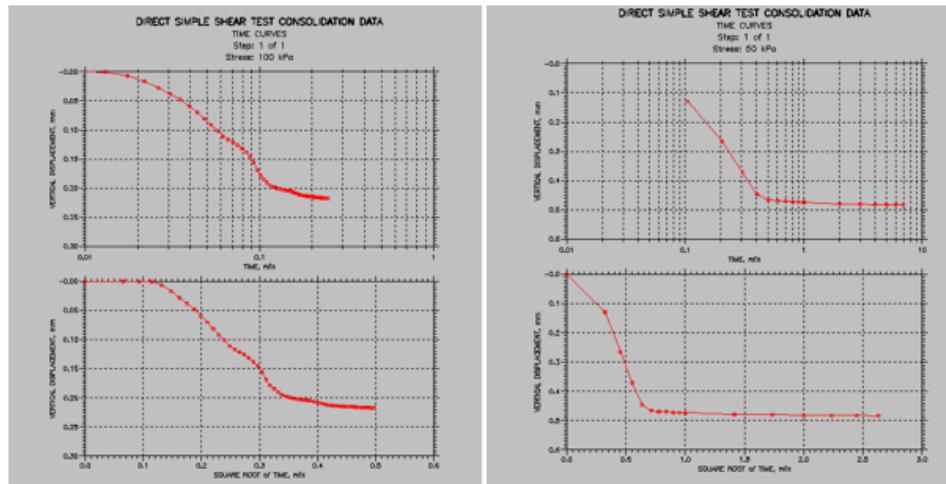


Figure 3.23. Typical Report for Consolidation Phase.

Then after the consolidation phase, the shearing phase starts. Here the material is shear by applying the cyclic shear stress vertically on the material. A typical output report can be seen in Figure 3.25.

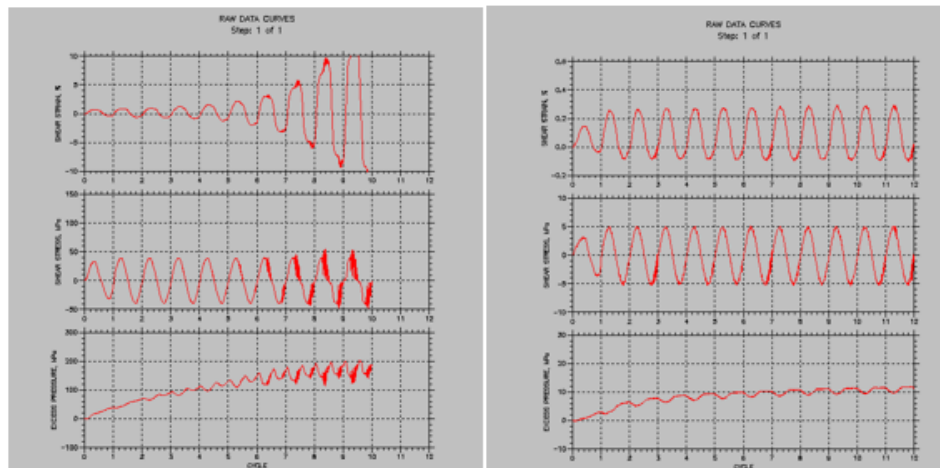


Figure 3.24. Typical Report for Shearing Phase.

Moreover, this study is conducted within a strain range that it could provide necessary information about some possible geotechnical applications. A chart is given Figure 3.25 about the shear strain ranges for different geotechnical applications and also with which apparatuses these ranges can be tested. As in this study the resonant column or bender is not used, strains smaller than 0.001 will not be investigated.

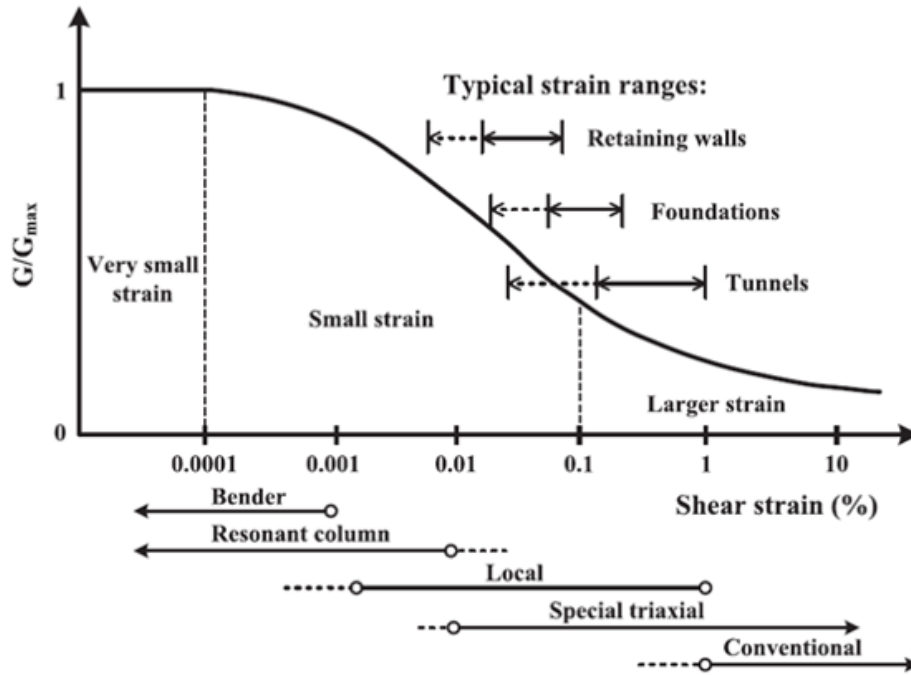


Figure 3.25. Typical Strain Ranges and tests for These Ranges (After Atkinson and Sallfors 1991; Mair 1993; Ishihara, 1996; Sawangsuriya *et al.*, 2005).

3.3.1. Damping Ratio

Damping ratio is a measure of energy dissipation and it increases with the increasing magnitude of cyclic shear strain as shown in the Figure 3.18. It is also seen that its value is proportional to the area inside the hysteresis loop in the same figure. Damping curves for a numerous range of soil types have been developed by some investigators (e.g., Seed *et al.*, 1986). It is known that damping curves are within a relatively narrow range for most cohesionless soils, as the variation of dynamic curves with change in soil properties is small for such soils.

3.3.1.1. Calculation of Damping Ratio. For the calculation of damping ratio, each loop which is obtained from the cyclic direct simple shear tests are treated. For that process, the definition of the term leads us to the formula given below:

$$D = \frac{\Delta W}{4\pi W} \quad (3.2)$$

here, D is the damping ratio (It may also be represented as), W is the maximum stored energy, ΔW is the energy loose per cycle represented by the area enclosed inside the hysteresis loop. And it leads:

$$D = \frac{W_D}{4\pi W_S} = \frac{1}{2\pi} \frac{A_{loop}}{G_{sec} \gamma_c^2} \quad (3.3)$$

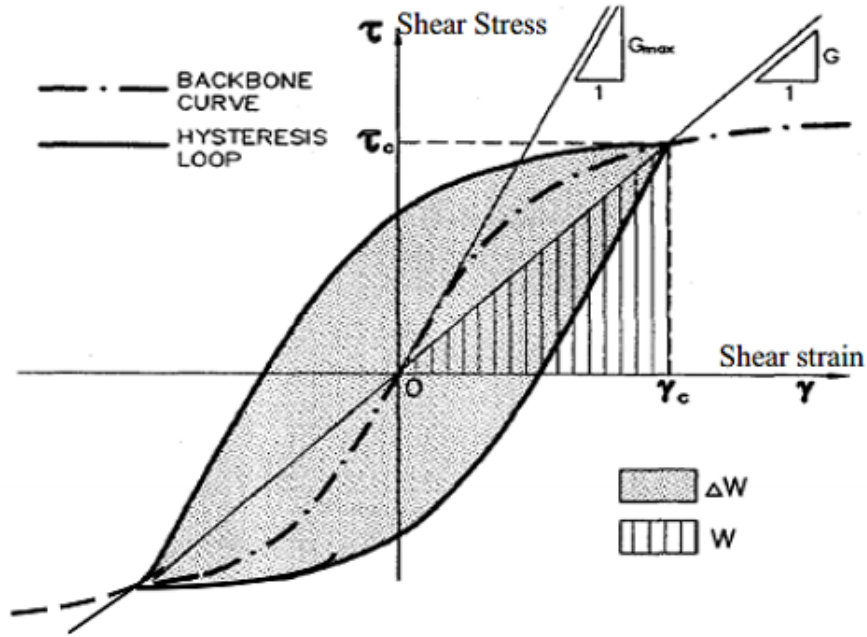


Figure 3.26. Use of Hysteresis Loop for Calculation of Damping Ratio (Kramer, 1996).

As explained in the introduction part, for fly ash pellets we expect a greater value of energy dissipation. This phenomenon can be explained with its low unit weight and greater void ratio. For questioning this claim, it is proper to use the damping ratio values of dry sand as comparison. A graph of G/G_{max} - damping ratio (provided by Khouri) is given in Figure 3.27.

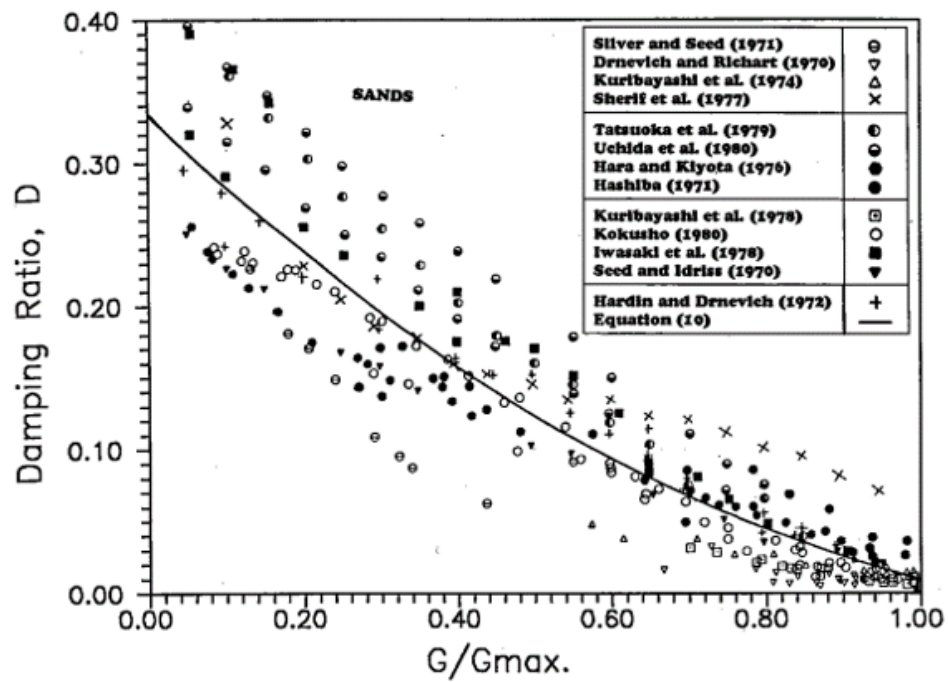


Figure 3.27. Damping Ratio versus G/G_{max} for sands (Khoury, 1984).

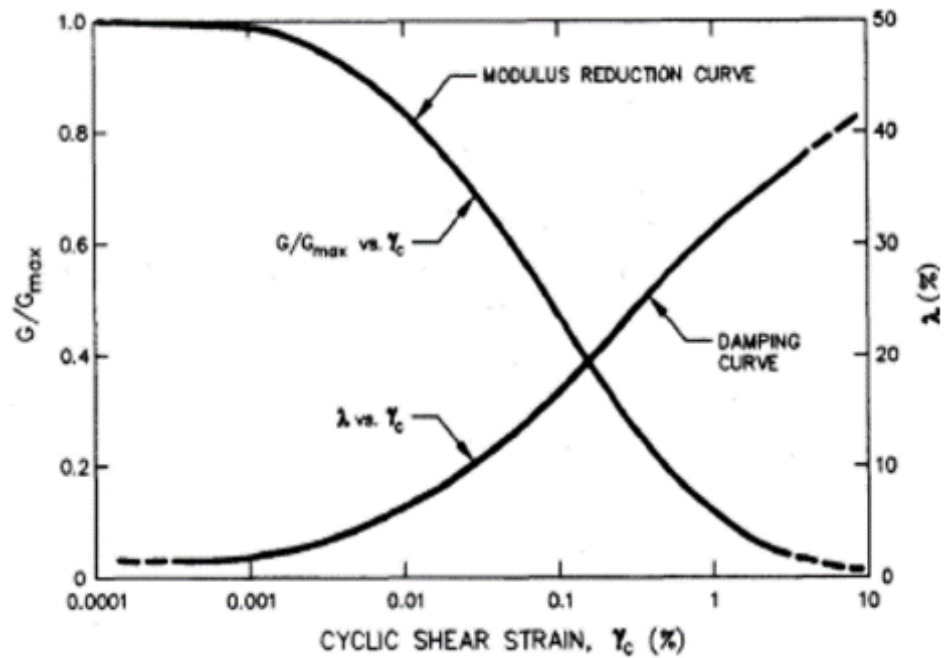


Figure 3.28. Shear Modulus Reduction and Equivalent Damping Ratio Curves (Kramer, 1996).

3.3.1.2. Calculating Damping Ratio by Using Cyclic Simple Shear Test Results. The data obtained from the simple shear test results are evaluated to have the area of the hysteresis curve for calculating damping ratio and shear modulus in two methods.

3.3.1.3. Finding the Area Inside of the Hysteresis Loop. As it is explained in the previous chapter, to obtain the damping ratio values, the area defined has to be computed. To realize this, some methods are developed. In this study, the software MATLAB is used to obtain the area inside the loops. It is a fast method and gives sufficiently accurate results. Also, it is much easier to work with many loops at the same time as the results are based on the data not the curve itself. Basically, a curve is fitted to the data introduced to the program. The software allows different approaches to fit a curve as depending on the accuracy and the boundary conditions. Then the program integrates this equation and it gives the area underneath as shown in the Figure 3.30.

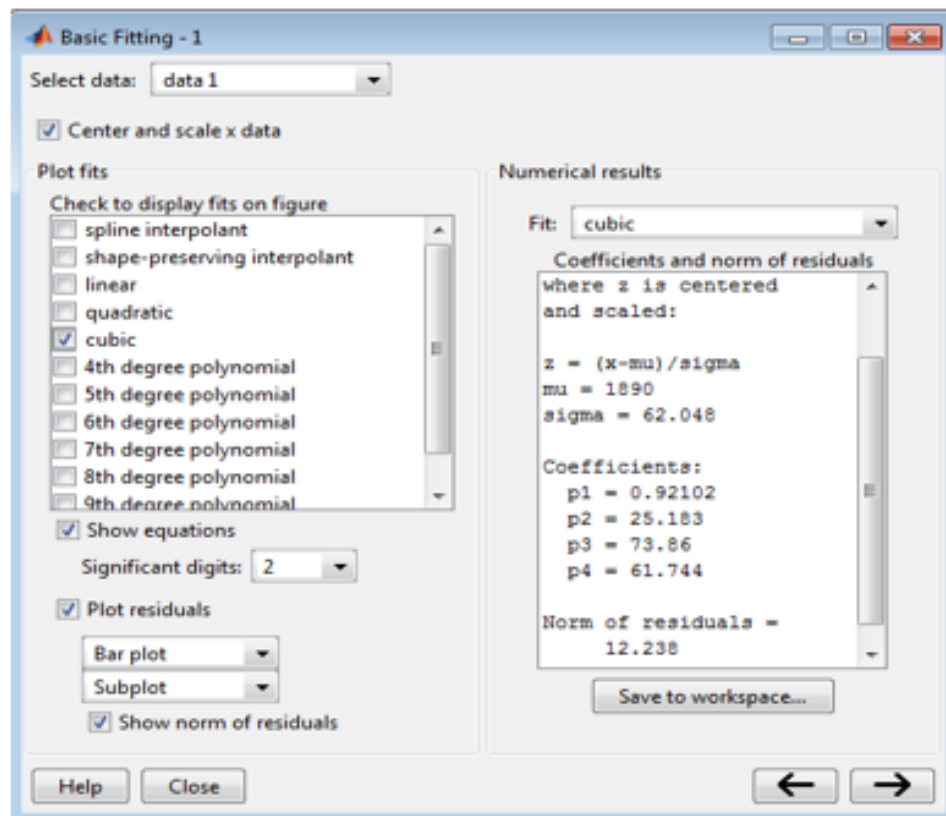


Figure 3.29. The Selection of Curve Fitting Alternatives.

$$Area = \int_0^{x_m} f(x) dx = \lim_{\Delta x \rightarrow 0} \sum_{i=1}^N f_i(x) \Delta x \quad (3.4)$$

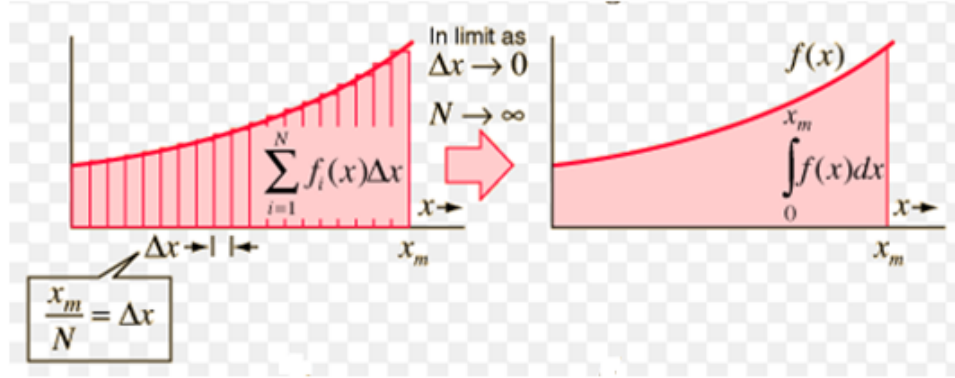


Figure 3.30. The Mathematical Background of Finding the Area Under a Curve.

3.3.2. Secant Modulus

When a cyclic loading is experienced by a soil particle in an ideal continuum (without the disturbance of adjacent structures etc.) it will behave as in the figure below and exhibit a hysteresis loop as shown in the Figure 3.33. In this study, as the results of DSS tests, the hysteresis loops are generated. The principal parameters defining the characteristics of these loops, their inclination and their breadth, are measured. The inclinations of these loops are defined as tangent shear modulus and it is used as the main parameter to represent the stiffness of the soil. In addition to this, obviously tangent shear modulus (G_{tan}) differs throughout the different shear strain values of a cycle of loading. So another parameter named as secant shear modulus (G_{sec}) and is used since it gives the average value over the entire loop.

$$G_{sec} = \frac{\tau_c}{\gamma_c} \quad (3.5)$$

where, τ_c is the shear stress, γ_v is shear strain amplitudes

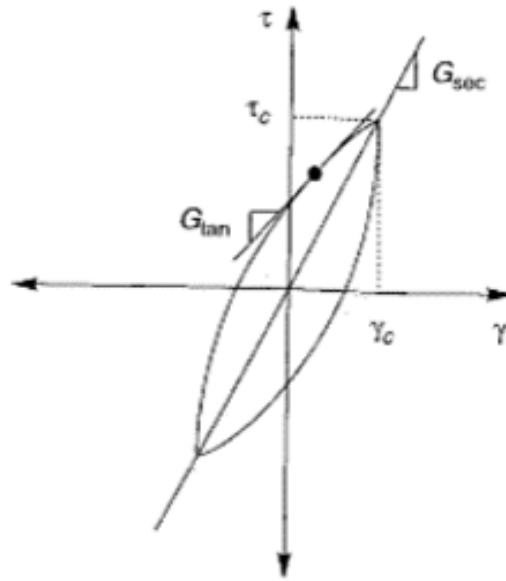


Figure 3.31. Typical Hysteresis Loo (Kramer, 1996).

The same setup for the previous chapter is used to compute the secant modulus values. By using the raw data, a curve is fitted by using MATLAB. Then the tangent of the line connecting the origin point to the desired shear strain value is calculated. The secant shear modulus values are used as the tangent shear modulus values are changing at every strain value whereas the secant shear modulus value is giving an average value for each curve at the hysteresis loops.

3.3.3. Triaxial Shear Test

In this study, the GeoComp LoadTrac II and FlowTrac II system was used which consists of two automatic flow pumps and a computer controlled loading frame. This apparatus fully automates the conduction of UU, CU, CD and any possible stress path triaxial tests on soils. Once a soil sample is in place, and the test conditions are selected, the system will run the entire triaxial test from start to finish. This system is operated by software which automates the initialization, saturation, consolidation (isotropic, anisotropic, or (K_o) and shear phases of the test.



Figure 3.32. Fully Automated Triaxial Testing Apparatus (Abatkon, 2012).

The main part of the testing apparatus, the loading frame which is also called load trac, has two functions. It controls the load applied on the specimen as well as measures the displacement during the test. Automatic flow pumps, intelligent units composed of a flow pump, a pressure sensor and a control board, regulate pressure and volume cell and sample. Flow tracs house precision micro stepper motors that move pistons in cylinders filled with water. Pressure transducers determine the signals being sent to the stepper motor and the number of motor steps is used to calculate the volume changes. Pressure increments can increase and decrease in any pattern by any amount. The count readings from sensors are converted to engineering units using the calibration factors.

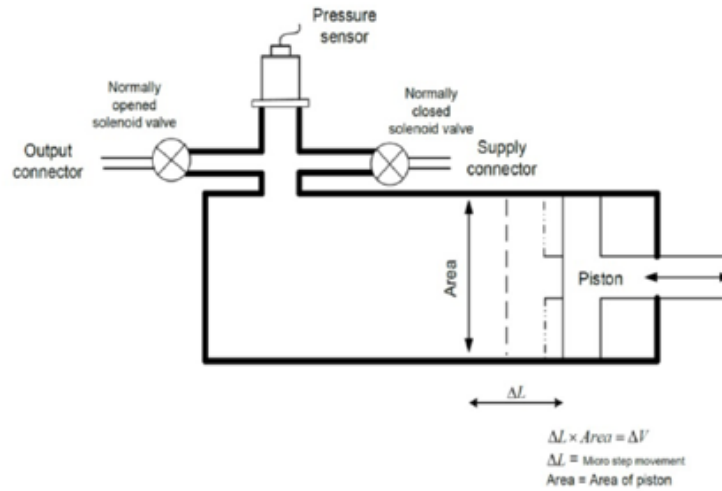


Figure 3.33. Schematic Representation of Flow Trac Unit (Abadkon, 2012).

3.3.3.1. Triaxial Testing Sample Preparation. The test specimen itself is prepared firstly from a sample of soil of sands and the fly ash pellets before placing into the triaxial cell. To obtain a larger relative density values, the samples were prepared with the air pluviation technique by using a funnel, together with a tube. Sand falls through the funnel inside the membrane from a constant height. The height is decided according to the needed relative density. The higher the falling height gets, the denser the specimens become. Also vibration was applied in order to achieve denser specimens.

3.3.3.2. Triaxial Testing Procedure. As explained before, GeoComp LoadTrac II and FlowTrac II used in this study to conduct the triaxial tests. It is conducted basically in four stages which are:

- (i) Initialization: During initialization phase small magnitudes of horizontal stress, vertical stress and pore pressure are applied to the specimen for about 20 minutes. This phase controls the propriety of apparatus functions.
- (ii) Saturation Phase: In saturation phase cell and pore pressures are raised incrementally while effective stress on the specimen is kept constant in order to achieve the maximum saturation ratio. According to ASTM (D-7181) pore pressure parameter (B ratio) is obtained from the following equation:

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad (3.6)$$

where Δu is the change in the specimen pore pressure that occurs when cell pressure changes and specimen drainage valves are closed and $\Delta \sigma_3$ is the change in cell pressure. A minimum value of 0.95 was achieved for all experiments.

- (iii) Consolidation Phase: The apparatus consolidates the specimen under K_0 condition. It automatically prevents the lateral straining by adjusting cell pressure. At rest earth pressure coefficient of sand samples could be monitored during the whole consolidation phase and it is calculated as:

$$K_0 = \frac{\sigma'_h}{\sigma'_v} \quad (3.7)$$

where σ'_h is the effective horizontal stress and σ'_v is the effective vertical stress.

- (iv) (iv) Shearing Phase: In this stage samples were sheared under drained conditions by increasing the vertical stress with the constant strain rate of $(2.5 \times 10^2)\%$ /min. According to ASTM (D- 7181) shear strain rate has been obtained from the following equation.

$$\dot{\epsilon} = \frac{4\%}{10t_{90}} \quad (3.8)$$

Where $\dot{\epsilon}$ is the shear strain rate and t_{90} is the time corresponding to 90% of primary consolidation. The maximum axial strain reached during shearing was 18% and the critical state of each sample was discernible.

4. RESULTS

In this chapter the direct simple shear results will be investigated. The results obtained from DSS test results are studied and the values to represent some behavior characteristics under cyclic loads and the liquefaction susceptibility are investigated. These studies are conducted for normal sands as well as the fly ash pellets and some significant differences or similarities are presented to distinguish the characteristics of fly ash pellets.

4.1. Shape Factor Analysis

The parameters used in this study are sphericity and roundness which is calculated as explained in the previous chapters.

The more proximate the roundness values get to 1, the less angularity is expected, meaning that the particles have more rounded corners. Besides as the sphericity approaches 1, the shape of the particle becomes more spherical. As can be seen in Table 4.1, both sphericity and roundness are the highest in well graded fly ash pellets and the lowest in Sile sand. Two different fly ash samples have nearly the same values but not exactly. This is due to the pelletization process, for obtaining different size of pellets, the pellet particles has to spend more time on the disc. This makes the particles more round and circular. So bigger pellet particles always have a greater sphericity and roundness values. The effect of this difference will be investigated in upcoming chapters.

Table 4.1. Average Sphericity and Roundness Values for All Sand Types.

Sand Type	Sphericity	Roundness
Sile Sand	0.66	0.74
FA Pellets with same gradation with Sile Sand (Well graded - SW)	0.8	0.85
Kilyos Sand	0.74	0.75
FA Pellets with same gradation with Kilyos Sand (Poorly Graded - SP)	0.8	0.85

4.2. Crushing Strength Test

The crushing strength test is done for each particle size available to demonstrate the average stress that the individual particles broke. The results are given in the table below.

Table 4.2. Crushing Strength Test Results.

Sieve size (mm)	Particle number	$D_{average}$ (mm)	crushing Strength (N)	average values for crushing strength	$\sigma_{average}$ (N/cm ²)	$\sigma_{average}$ (kPa)	
19	1	20.0	297.13	301.18	0.74	0.75	7.53
	2	20.0	301.15		0.75		
	3	20.0	305.18		0.76		
	4	20.0	289.21		0.72		
	5	20.0	313.24		0.78		
16	1	17.50	212.40	216.5	0.69	0.71	7.07
	2	17.5	214.45		0.70		
	3	17.5	216.50		0.71		
	4	17.5	218.55		0.71		
	5	17.5	220.60		0.72		
12.5	1	14.3	202.65	204.05	1	1	10.05
	2	14.3	204.46		1.01		
	3	14.3	204.16		1.01		
	4	14.3	205.42		1.01		
	5	14.3	203.56		1.00		
10	1	11.30	99.65	98.61	0.79	0.78	7.79
	2	11.3	97.46		0.77		
	3	11.3	99.46		0.79		
	4	11.3	100.01		0.79		
	5	11.3	96.48		0.76		
8	1	9	64.57	64.22	0.8	0.79	7.93
	2	9	66.49		0.82		
	3	9	63.13		0.78		
	4	9	61.48		0.76		
	5	9	65.46		0.81		
6	1	7.2	41.87	41	0.82	0.8	8.02
	2	7.2	40.99		0.80		
	3	7.2	42.48		0.83		
	4	7.2	39.92		0.78		
	5	7.2	39.76		0.78		
5	1	5.5	38.48	38.4	1.26	1.26	12.58
	2	5.5	37.49		1.23		
	3	5.5	39.02		1.28		
	4	5.5	39.87		1.31		
	5	5.5	37.15		1.22		

The results presented above are compared to those of Danyıldız (2007) given in literature review part. Danyıldız found the crushing strength around 246 N for pellets with 1.24 cm of diameter. This size corresponds 12.5 mm of sieve size and the result of the test is 204 N. Also the results found by Arslan are in the same range of the results presented above. It can be said that the pellets used in this study are more crushable

than those of the previous study.

4.3. Monotonic Triaxial Test Results

CD triaxial tests were conducted on 4 different sample samples in both loose and dense state. By using the Mohr circles generated from the test, internal friction angles are obtained and given in the Table 4.3.

Table 4.3. Internal Friction Angles of Samples.

Samples	Relative Density	Internal Friction Angle
Fly Ash Pellet (Kilyos Sand Gradation)	0.85	34.9°
Fly Ash Pellet (Kilyos Sand Gradation)	0.21	31.3°
Kilyos Sand	0.88	38.8°
Kilyos Sand	0.24	36.6°
Fly Ash Pellet (Sile Sand Gradation)	0.9	38.8°
Fly Ash Pellet (Sile Sand Gradation)	0.18	35.7°
Sile Sand	0.82	40.4°
Sile Sand	0.12	37.6°

4.4. Secant shear modulus (G_{sec}) and normalized shear modulus (G_{sec}/G_{max})

When a cyclic loading is experienced by a soil particle in an ideal continuum (without the disturbance of adjacent structures etc.) it will behave as in the figure below and exhibit a hysteresis loop as shown. In this study, as the results of DSS tests, the hysteresis loops are generated. The principal parameters defining the characteristics of these loops, their inclination and their breadth, are measured. The inclinations of these loops are defined as tangent shear modulus and it is used as the main parameter to represent the stiffness of the soil. In addition to this, obviously tangent shear modulus (G_{tan}) differs throughout the different shear strain values of a cycle of loading. So another parameter named as secant shear modulus (G_{sec}) and is used since it gives the average value over the entire loop.

$$G_{sec} = \frac{\tau_c}{\gamma_c} \quad (4.1)$$

where, τ_c is the shear stress, γ_c is shear strain amplitudes

In this study, shear modulus values are calculated and presented as given below. Firstly, it is held in two groups with different consolidation pressure. Then the values obtain by using different samples and different loading conditions are introduced and the results are discussed.

4.4.1. The Results for 50 kPa Consolidation Pressure

The results for 50 kPa consolidation pressure will be given for different materials and loading conditions below.

4.4.1.1. Sile Sand and the Fly Ash Pellet Sample with Same Gradation. The results obtained in Cyclic DSS tests are studied and G_{sec} values are calculated as explained in previous chapters. Their variations against shear strain are presented in figures below. Generally the modulus values are presented as normalized by G_{max} in the literature, but since G_{max} has to be computed by using shear wave velocities at the r, in this study that part will be left out.

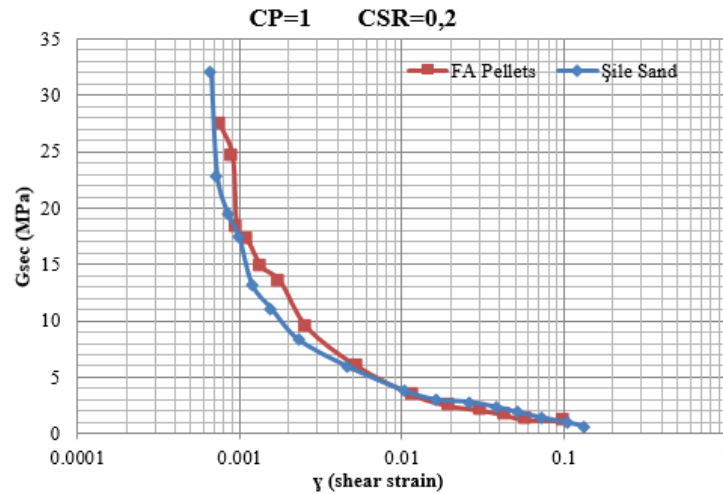


Figure 4.1. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CRS=0.2).

As seen in the Figure 4.2, at CP=1 and CSR=0.2 loading condition, the shear

modulus reduction curves of Sile Sand, shows a typical behavior for sands. The modulus values are decreasing as the shear strains are increasing. After 1% shear strain threshold, the modulus values are getting stabilized. This trend is also present at the same gradation fly ash pellet sample as given in the Figure 4.3.

The fly ash pellets with same gradation of Sile Sand exhibit a similar behavior. It has smaller shear modulus values in comparison with Sile Sand at the small strains but as the strain values increase the curves nearly overlapping. Besides, for the shear strain values between about 0.1% and 1%, it is seen that fly ash pellets shows even a better performance than Sile Sand.

Although the two different sets of reduction curves shows the same trend of decrease, the shear modulus values tend to be slightly smaller for fly ash pellets with the same gradation for CP=1 and CSR=0.2 loading condition.

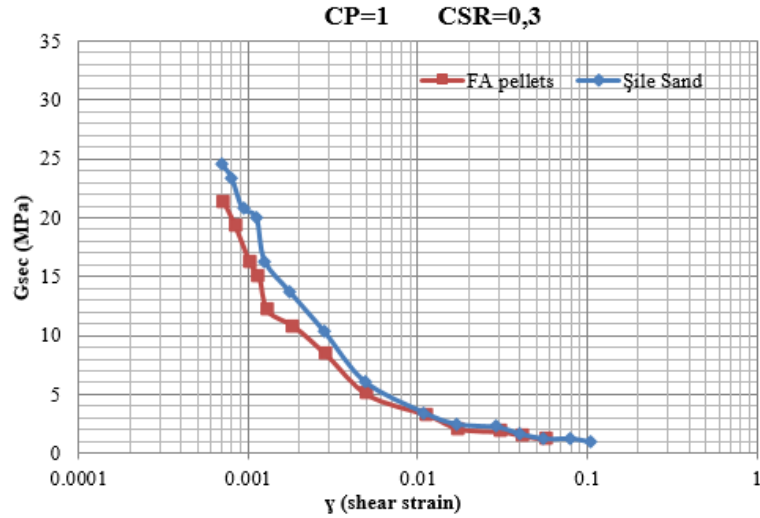


Figure 4.2. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).

The same behavior is seen at the shear modulus reduction curves of Sile Sand and fly ash pellets with same gradation at CSR=0.3 and CP=1 sec. The shear modulus values decrease as the strain is increasing. So here is seen again that the fly ash pellets that are produced in this study exhibit sand-like behavior under cyclic loads. Moreover, both curves start at a smaller shear secant value. So it can be said that cyclic stress

ratio and the secant modulus are inversely proportional. As CSR value is increasing, the secant modulus values would decrease. Even if the decrease is visible for both curves, it is more pronounced for sand. Besides, the better behavior of fly ash pellets for some strain ranges is not valid anymore. Along the entire curve, the shear modulus values for Sile Sand is clearly higher than those of fly ash pellets. But it also has to be noted that this difference is not that much pronounced and within the tolerable limits.

The third loading is with the same CSR value but the cycles that are applied on specimen have periods of 2 sec instead of 1. For this loading condition, again the Sile Sand has higher values throughout the entire shear strain range than the fly ash pellets. But as overall, the effect of change of cycle period on the shear modulus reduction curve is not significant.

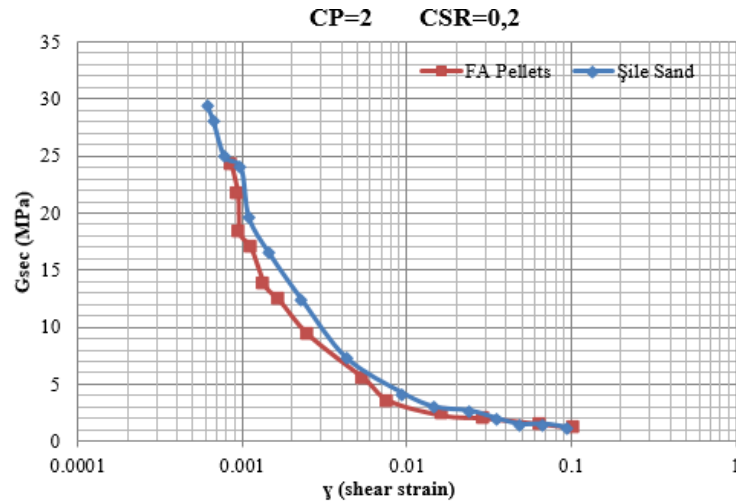


Figure 4.3. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=2 CSR=0.2).

4.4.1.2. Kilyos Sand and Fly Ash Pellets with Same Gradation. The same observation on the reduction curves of Kilyos sand can be seen. The trend of decrease is still follows the same path. It decreases sharply around 0.1% strain levels and then stabilizes after 1% strain levels. But the strain modulus values are smaller than 20 MPa at the same loading conditions. The curve that belongs to the fly ash pellet sample with the same gradation gives a different shape as seen in Figure 4.5. The shear strain values are much bigger than the ones in sand sample.

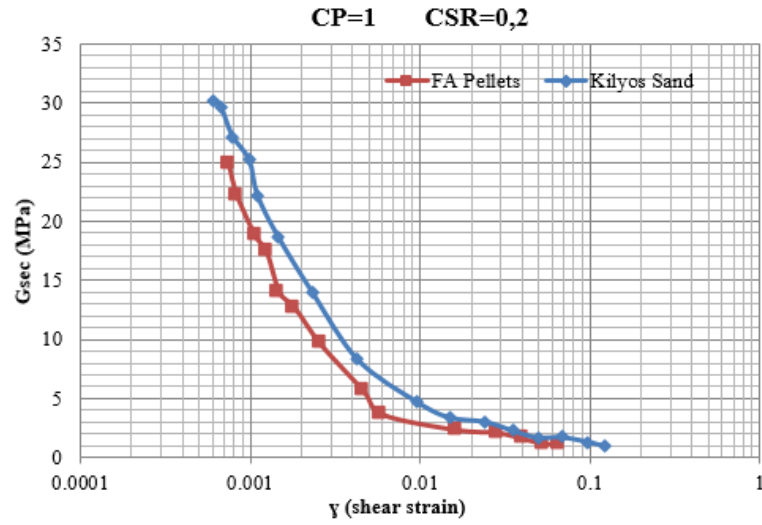


Figure 4.4. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.2).

Secondly, for CSR=0.3 and CP=1 loading conditions, same trend of decreasing through the increasing shear strain whereas the starting values of both curves are lower than the CSR=0.2. So as stated for Sile Sand, it can be said that increasing CSR values decrease the secant shear modulus values. This decrease is more accented then the one experienced in Sile Sand.

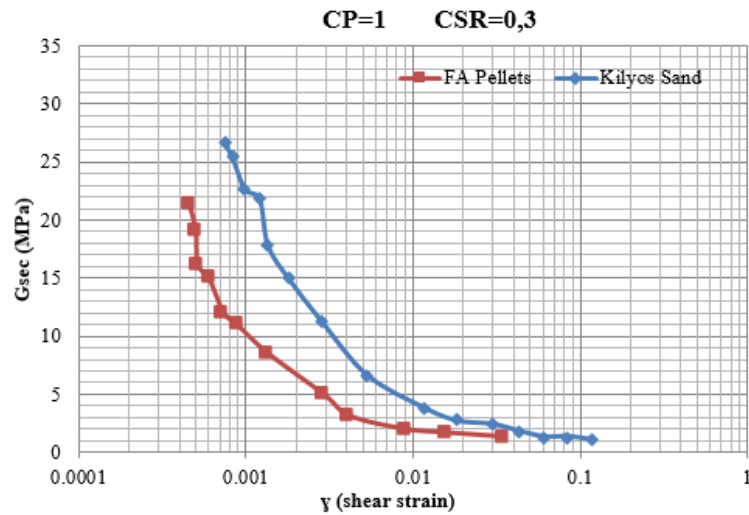


Figure 4.5. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).

Lastly for the curves of $CR=0.2$ and $CP=2$ seconds, the same trend of decrease of shear modulus is seen clearly in the same manner.

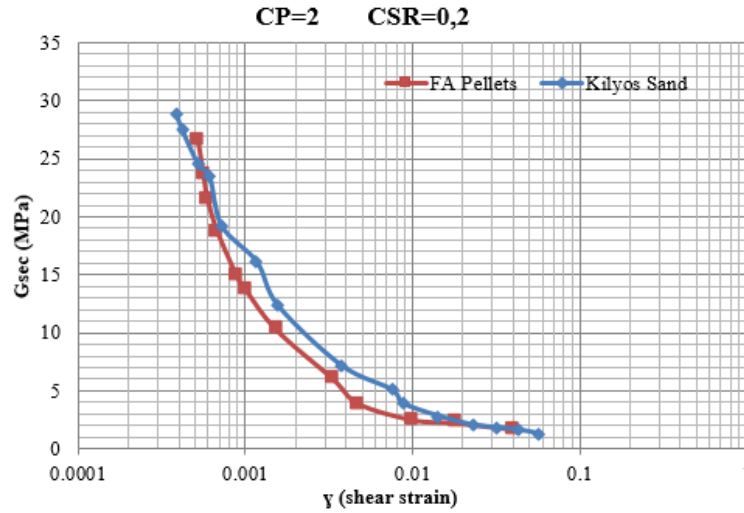


Figure 4.6. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions ($CP=2$ $CSR=0.2$).

So it is seen that shear modulus decreases with the shear strain increase. This trend starts linearly and then turns into a curvature. Then as seen clearly in Figure 4.5 and Figure 4.2, Sile Sand has bigger shear modulus values than Kilyos Sand. It can be said that the gradation of the material plays an important role on its stiffness behavior. As explained in material properties, Sile Sand is well graded, and this helps material to have a stiffer behavior.

4.4.2. The Results for 100 kPa Consolidation Pressure

The results for 100 kPa consolidation pressure will be given for different materials and loading conditions below.

As a different case from 50 kPa test, here it is also has to be considered the amount of particles which experience some degree of crushing during the test. As it leaves a remarkable change of behavior for material, the grain size analysis before and after the test under 100 kPa consolidation pressure are also given in the following pages.

4.4.2.1. Sile Sand and Fly Ash Pellets with Same Gradation. As seen in the Figure 4.8, at $CP=1$ and $CSR=0.2$ loading condition, the shear modulus reduction curves of Sile Sand, shows a typical behavior for sands, not differing a lot from 50 kPa consolidation. The modulus values are decreasing as the shear strains are increasing. After 1% shear strain threshold, the modulus values are getting stabilized. This trend is also present at the same gradation fly ash pellet sample as given in the Figure 4.8.

The fly ash pellets with same gradation of Sile Sand exhibit a similar behavior. It has smaller shear modulus values in comparison with Sile Sand at the small strains but as the strain values increase the curves nearly overlapping. Besides, for the shear strain values between about 0.1% and 1%, it is seen that fly ash pellets shows even a better performance than Sile Sand.

Although the two different sets of reduction curves shows the same trend of decrease, the shear modulus values tend to be slightly smaller for fly ash pellets with the same gradation for $CP=1$ and $CSR=0.2$ loading condition.

The effect of cyclic stress ratio is also visible seen the Figure 4.8. The shear modulus values are significantly lower at higher CSR values. This decrease is much more visible at sand specimens. The fly ash sample is nearly not affected by the increase of CSR. Even though they are both having some different characteristic under different loading condition, the general variation of secant modulus against shear strain stays somehow unchanged.

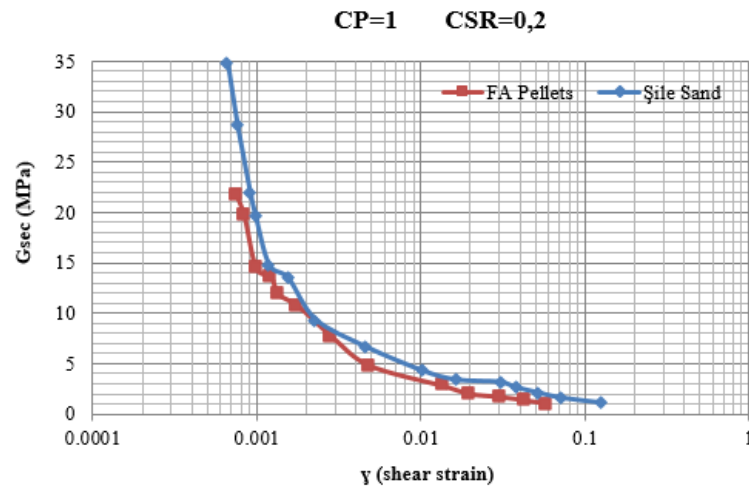


Figure 4.7. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.2).

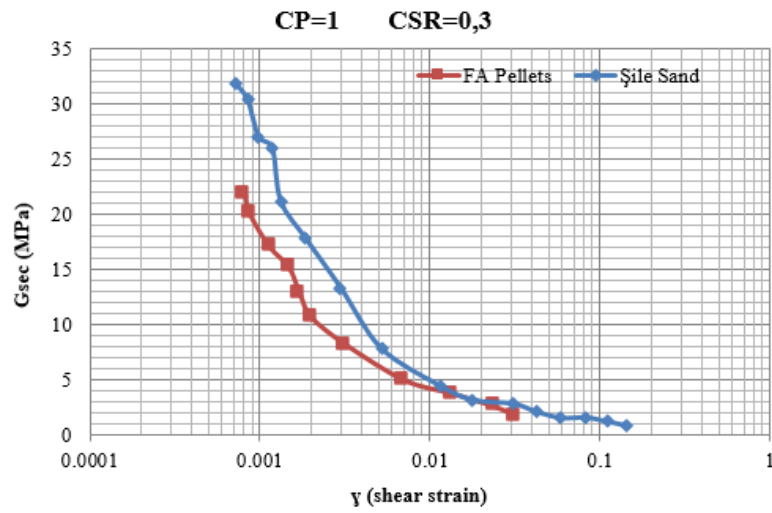


Figure 4.8. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions (CP=1 CSR=0.3).

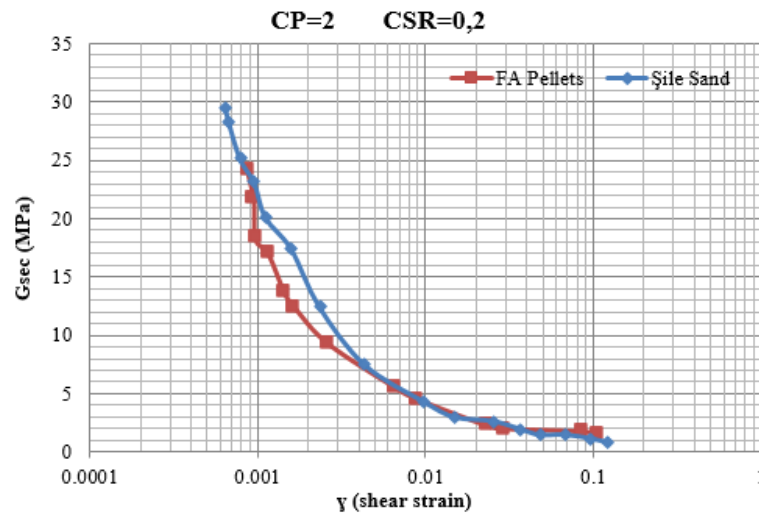


Figure 4.9. Shear Modulus Reduction Curves of Sile Sand and Same Graded Pellets at Given Loading Conditions ((CP=2 CSR=0.2).

Lastly for the CSR=0.2 and CP=2 seconds loading conditions, the same behavior is clearly seen whereas there is a slight decrease for both values. Even though the decrease is visible for both curves, it is more pronounced for sand. Besides, there is not any strain range that the fly ash pellets exhibits higher stiffness behavior anymore. Along the entire curve, the shear modulus values for Sile Sand is clearly higher than those of fly ash pellets. But it also has to be noted that this difference is not that much pronounced and within the tolerable limits.

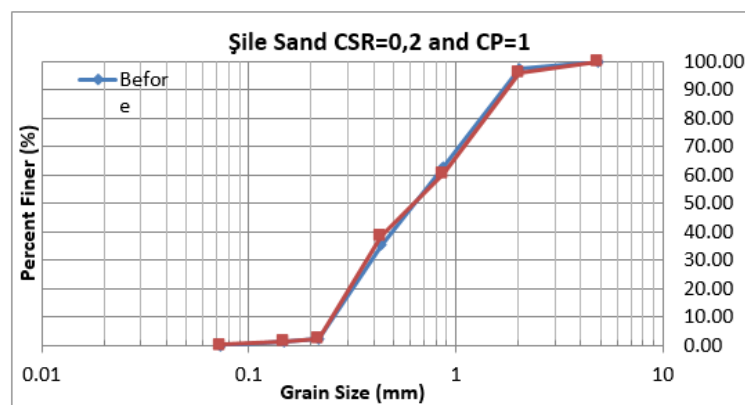


Figure 4.10. Grain Size Analysis After and Before the Test at CSR=0.2 and CP=1 for Sile Sand.

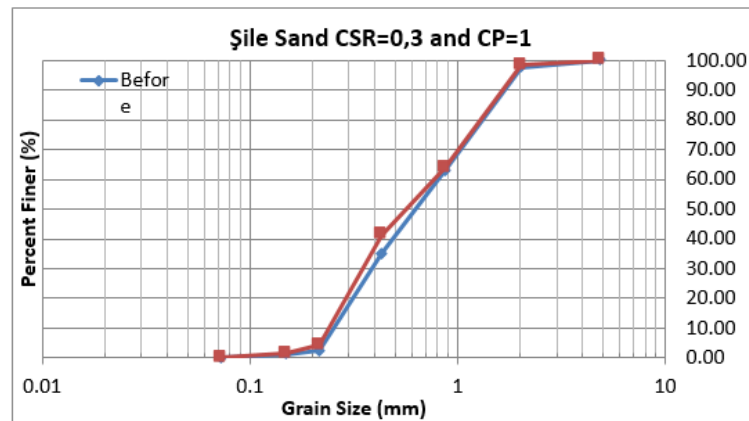


Figure 4.11. Grain Size Analysis After and Before the Test at CSR=0.3 and CP=1 for Sile Sand.

The third loading is with the same CSR value but the cycles that are applied on specimen have periods of 2 sec instead of 1. For this loading condition, again the Sile Sand has higher values throughout the entire shear strain range than the fly ash pellets. But as overall, the effect of change of cycle period on the shear modulus reduction curve is not significant. It is also noted for Sile sand under 100 kPa consolidation pressure, as shown in the Figure 4.11 and 4.12, some degree of crushing is experienced but it's lower than 5%. But on the other hand, for fly ash pellets, crushing occurs at higher rates as shown in the Figure 4.13. Even though they experience a great crushing during the test, it is below 12%.

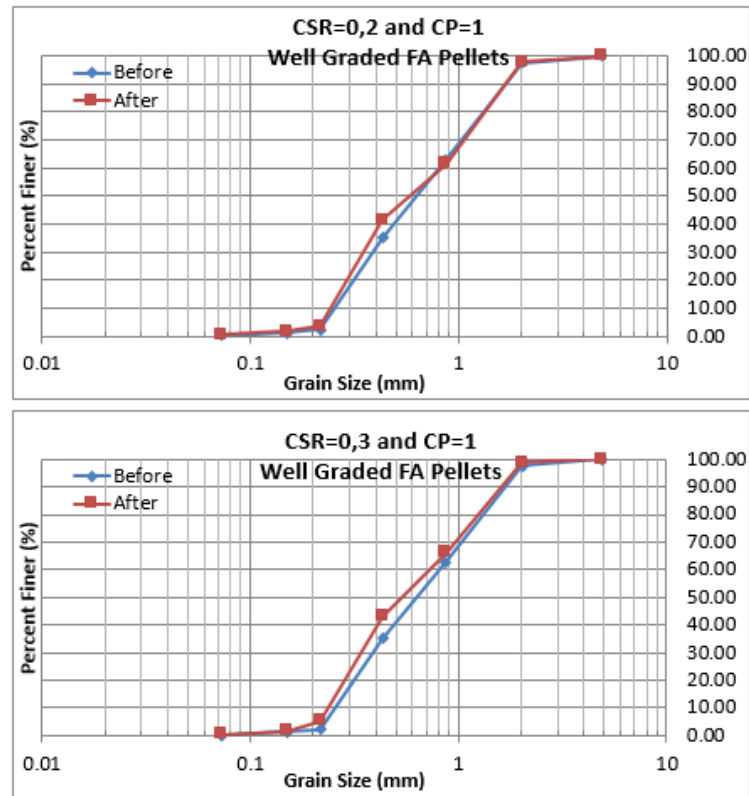


Figure 4.12. Grain Size Analysis After and Before the Test at CSR=0,3 and CP=1 for Well Graded FA Pellets.

4.4.2.2. Kilyos Sand and Fly Ash Pellets with Same Gradation. As seen in the Figure 4.17, at CP=1 and CSR=0.2 loading condition, the shear modulus reduction curves of Kilyos Sand, shows a typical behavior for sands. The modulus values are decreasing as the shear strains are increasing. After 1% shear strain threshold, the modulus values are getting stabilized. This trend is also present at the same gradation fly ash pellet sample as given in the Figure 4.11.

The fly ash pellets with same gradation of Kilyos Sand exhibit a similar behavior. It has smaller shear modulus values in comparison with Kilyos Sand at the small strains but as the strain values increase the curves nearly overlapping. Besides, for the shear strain values between about 0.1% and 1%, it is seen that fly ash pellets shows even a better performance than Kilyos Sand.

Although the two different sets of reduction curves shows the same trend of

decrease, the shear modulus values tend to be slightly smaller for fly ash pellets with the same gradation for CP=1 and CSR=0.2 loading condition.

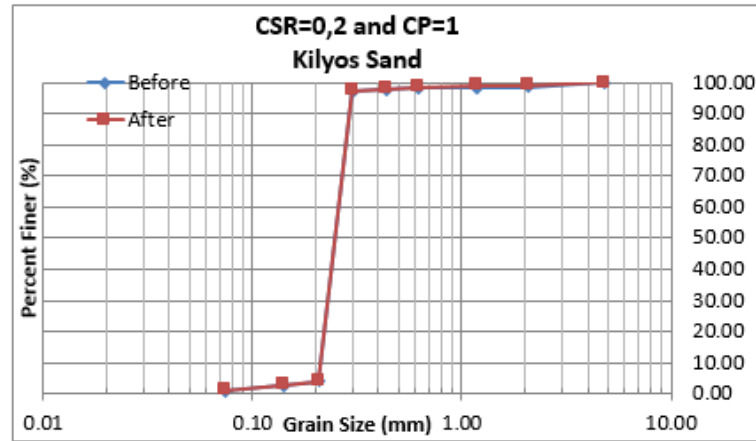


Figure 4.13. Grain Size Analysis After and Before the Test at CSR=0,2 and CP=1 for Kilyos Sand.

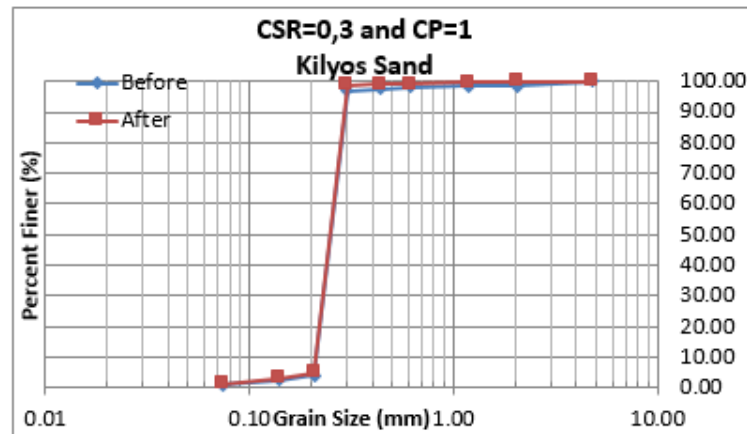


Figure 4.14. Grain Size Analysis After and Before the Test at CSR=0,3 and CP=1 for Kilyos Sand.

As shown in the Figure 4.14 and Figure 4.15, for Kilyos sand under 100 kPa consolidation pressure, some degree of crushing is experienced but it's lower than 5%. So it is considered as there is no crushing according to USCS. But on the other hand, for fly ash pellets at the same gradation, crushing occurs at higher rates as shown in the Figure 4.16. Even though they experience a great crushing during the test, it is below 12%. The crushing rates for poorly graded fly ash pellets are less than those for

well graded fly ash pellets. This can be explained as the size of pellets increase the crushing strength of individual pellets decrease according to its size. The individual crushing strength values are studied at previous chapters and it confirms this idea. As the well graded fly ash pellet sample contains bigger pellet particles, they crushed at lower stresses than the smaller particles.

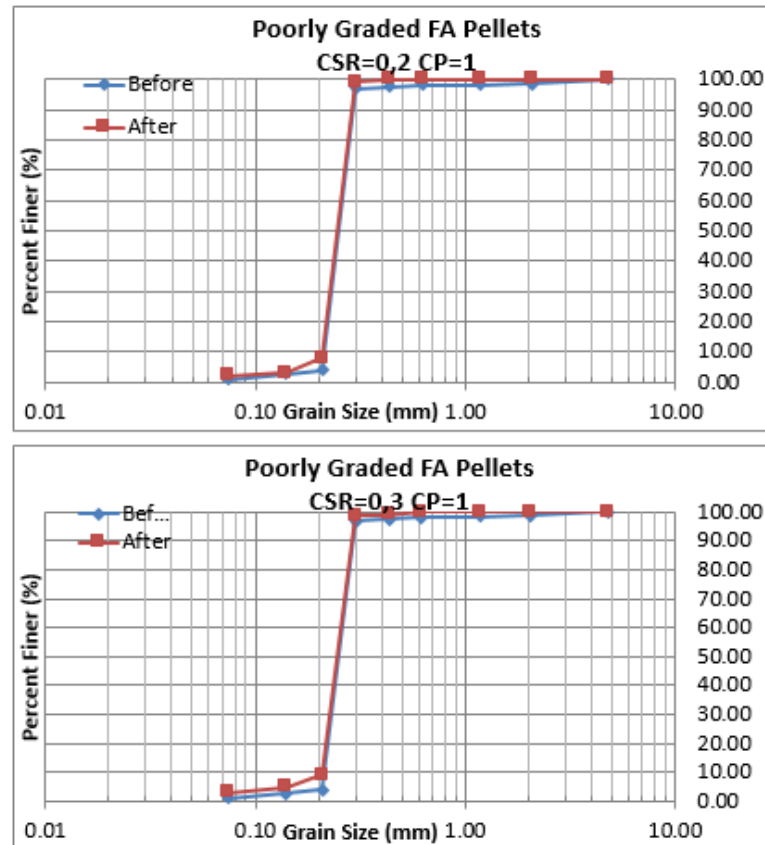


Figure 4.15. Grain Size Analysis After and Before the Tests for Kilyos Sand.

As seen in the Figure 4.17 and Figure 4.18, at $CP=1$ and $CSR=0.2$; $CSR=0.3$ and $CP=1$ loading conditions, the shear modulus reduction curves of Kilyos Sand, shows a typical behavior for sands. The modulus values are decreasing as the shear strains are increasing. After 1% shear strain threshold, the modulus values are getting stabilized.

The fly ash pellets with same gradation of Kilyos Sand exhibit a similar behavior. It has smaller shear modulus values in comparison with Kilyos Sand at the small strains but as the strain values increase the curves nearly overlapping. Besides, for the shear strain values between about 0.1% and 1%, it is seen that fly ash pellets shows even a

better performance than Kilyos Sand.

Although the two different sets of reduction curves shows the same trend of decrease, the shear modulus values tend to be slightly smaller for fly ash pellets with the same gradation for CP=1 and CSR=0.2 loading condition.

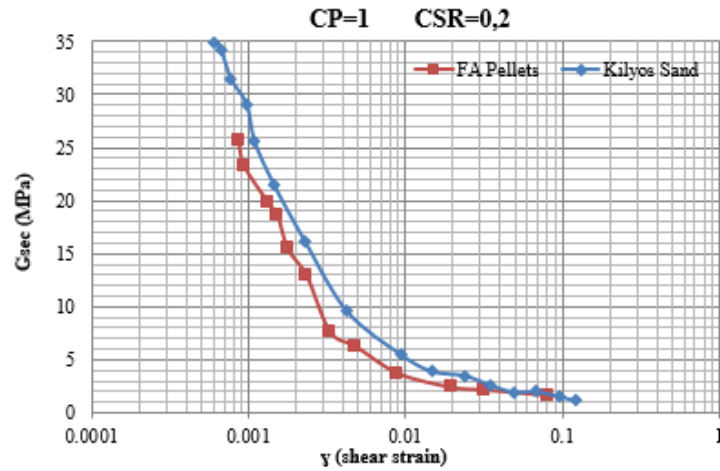


Figure 4.16. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2).

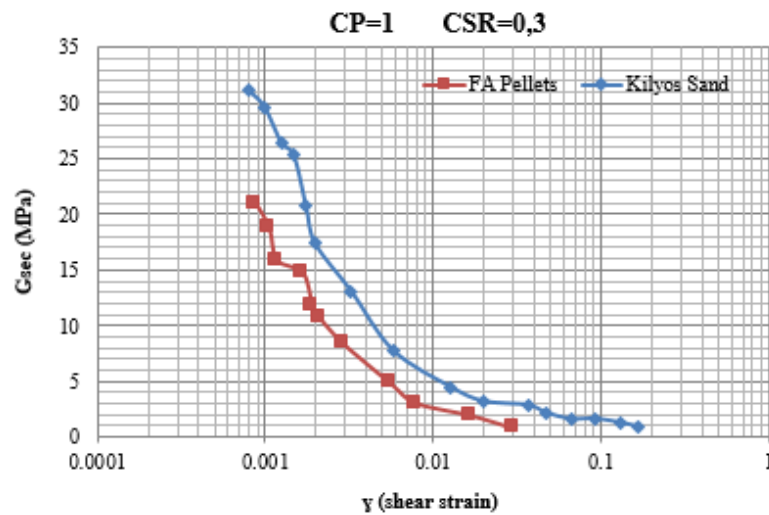


Figure 4.17. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3).

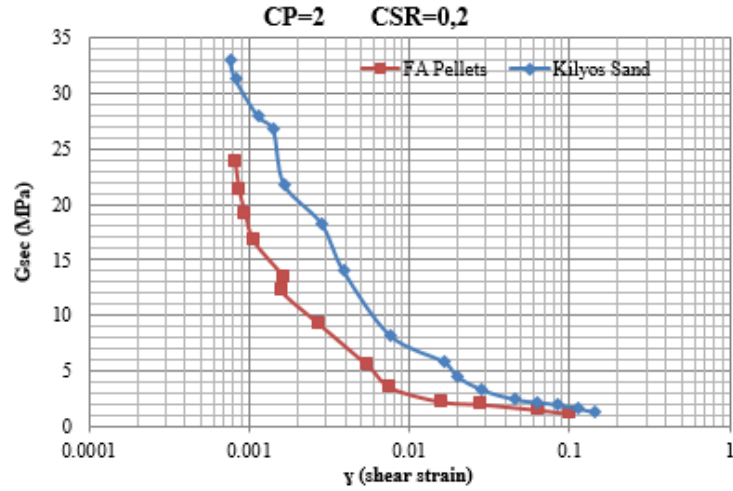


Figure 4.18. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2).

4.5. Damping Ratio

Stress waves in a soil substratum induce cyclic shearing and compressional loadings with decay of the amplitude away from the source, which is partly due to material losses. The energy transformation in the material is observed through its hysteretic behavior. Loops of varying amplitude and direction appear in the stress-strain plane, which is a major difference between considering cyclic loading of soils from monotonic loadings. In the laboratory also, during harmonic or triangular loading, stress-strain curves form series of loops. The damping ratio quantifies those material losses. More than just a model parameter, its physical meaning is crucial for understanding cyclic soil behavior.

The most common way, among geotechnical engineers, to apprehend the energy loss of a soil submitted to cyclic loading is the damping ratio (D or ξ).

In this study damping ratio values are calculated and given in next chapters.

4.5.1. The Results for 50 kPa Consolidation Pressure

As it is held in two groups in the study of shear modulus values, the same classification will be applied for damping ratio values.

4.5.1.1. Sile Sand and the Fly Ash Pellets with the Same Gradation. The damping ratio values for both pellets and Sile Sand in the Figure below. The damping ratio values are increasing nonlinearly as the shear strain is increasing. As seen in the Figure, after 1% of strain the damping ratio values for fly ash pellets tend to be higher than the Sile Sand. For higher CSR values, it is seen that the damping ratio values are remarkably higher. So it is clear that as CSR increase the damping ratio is increasing remarkably. Increases of CSR also contribute to increase rate of damping ratio throughout the curve. As for CSR=0,2 the increase is around 10%, for CSR=0,3 the difference is more than 20%. The same effect can be also seen at the third graph. As the cyclic period is doubled, damping ratio values increased slightly too. Even though, it is not so remarkable for the small strain levels, it becomes more accented after certain shear strain values.

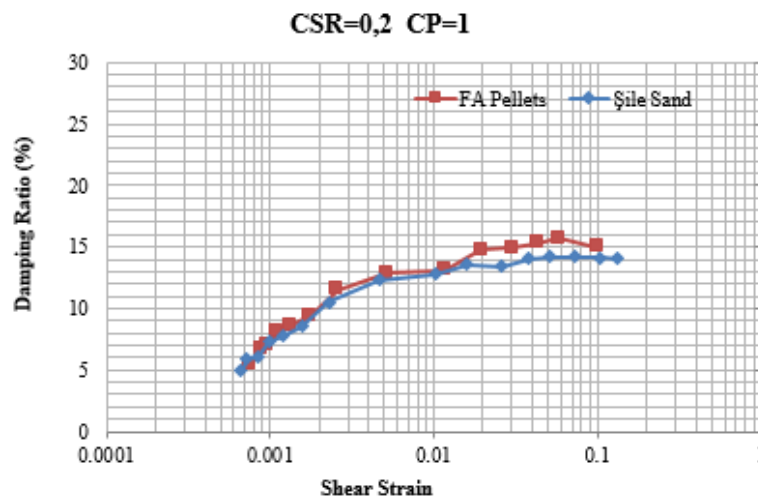


Figure 4.19. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2).

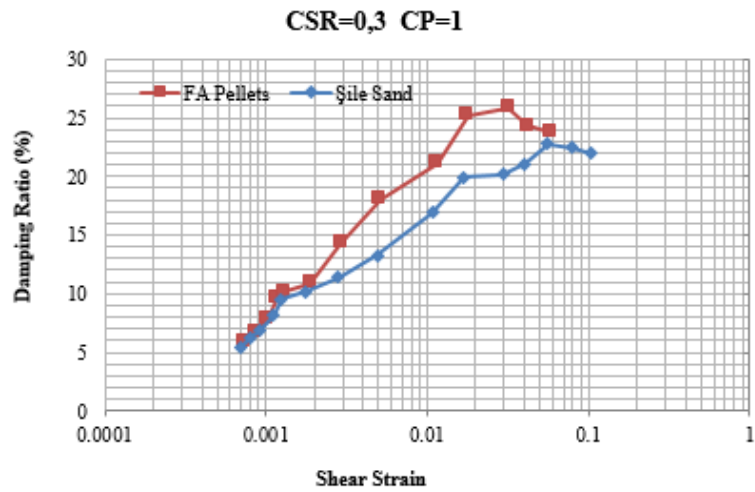


Figure 4.20. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3).

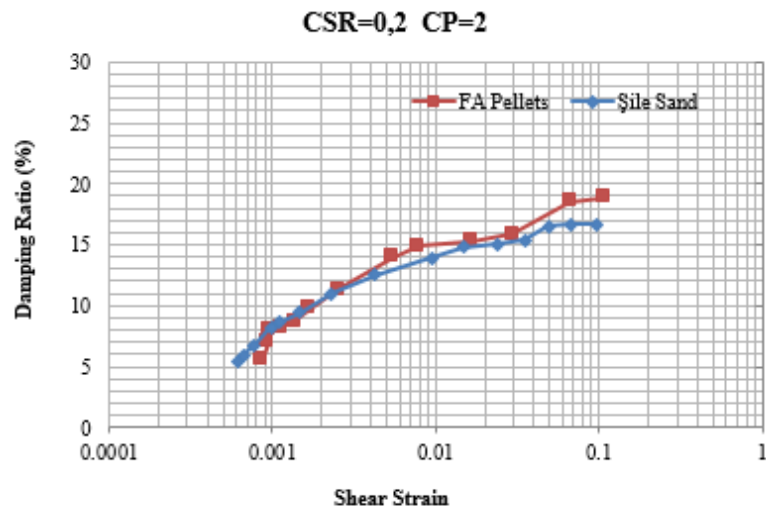


Figure 4.21. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2).

4.5.1.2. Kilyos Sand and the Fly Ash Pellets with Same Gradation. The same trend of increase of damping ratio as the strains are increasing is noted also for the Kilyos Sand. As seen in the Figure, after 0.1% of strain the damping ratio values for fly ash pellets tend to be higher than the Şile Sand. But it is not constantly increasing. After the strain levels of 4% the trend for fly ash pellets and sand particles differ remarkably. This can be due to the crushing of the fly ash pellets. Same observed as in

Sile Sand for higher CSR values, it is seen that the damping ratio values are remarkably higher. Increase of CSR also contribute to increase rate of damping ratio throughout the curve. As for CSR=0.2 the increase is around 10%, for CSR=0.3 the difference is more than 20%. The same effect can be also seen at the third graph. As the cyclic period is doubled, damping ratio values increased slightly too. Even though, it is not so remarkable for the small strain levels, it becomes more accented after certain shear strain values.

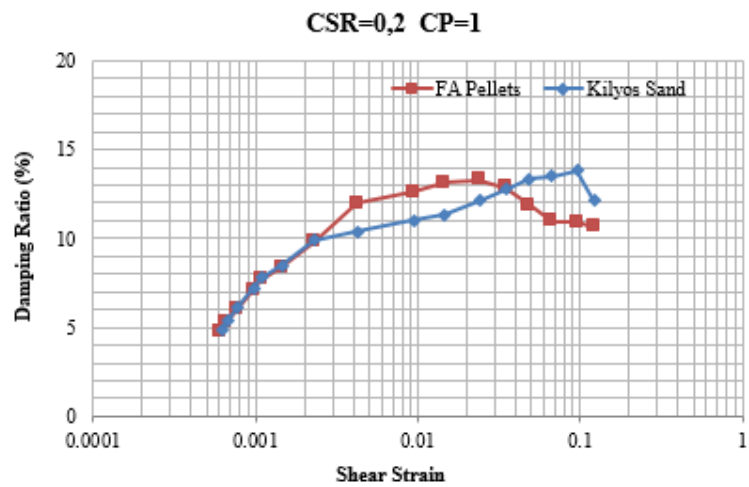


Figure 4.22. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3).

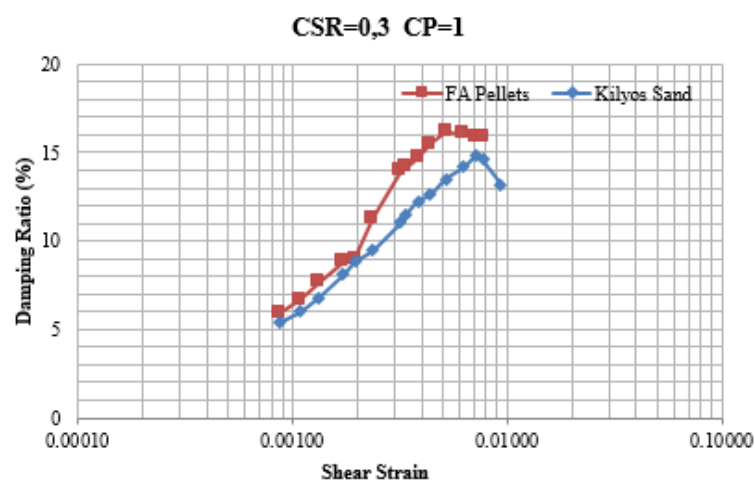


Figure 4.23. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3).

The same observation on the reduction curves of Kilyos sand can be seen. The trend of decrease is still follows the same path. It decreases sharply around 0.1% strain levels and then stabilizes after 1% strain levels. But the strain modulus values are smaller than 20 MPa at the same loading conditions. The curve that belongs to the fly ash pellet sample with the same gradation gives a different shape as seen in Figure. The shear strain values are much bigger than the ones in sand sample.

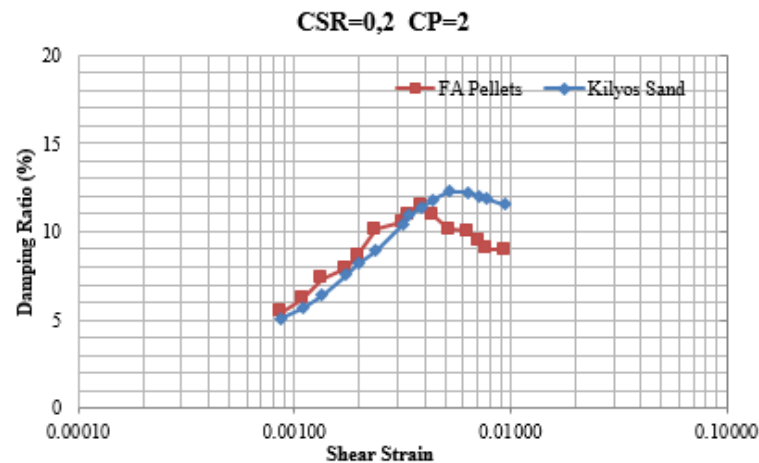


Figure 4.24. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2).

4.5.2. The Results for 100 kPa Consolidation Pressure

After the results of 50 kPa consolidation pressure are presented, now the tests done with 100kPa consolidation pressure will be given.

4.5.2.1. Sile Sand and the Fly Ash Pellets with the Same Gradation. The same trend of increase of damping ratio as the strains are increasing is noted also for the Sile Sand. As seen in the Figure, after 0.1% of strain the damping ratio values for fly ash pellets tend to be higher than the Sile Sand. But it is not constantly increasing. After the strain levels of 4% the trend for fly ash pellets and sand particles differ remarkably. This can be due to the crushing of the fly ash pellets. Same observed as in Sile Sand for higher CSR values, it is seen that the damping ratio values are remarkably higher. Increase of CSR also contribute to increase rate of damping ratio throughout

the curve. As for $CSR=0.2$ the increase is around 10%, for $CSR=0.3$ the difference is more than 20%. The same effect can be also seen at the third graph. As the cyclic period is doubled, damping ratio values increased slightly too. Even though, it is not so remarkable for the small strain levels, it becomes more accented after certain shear strain values.

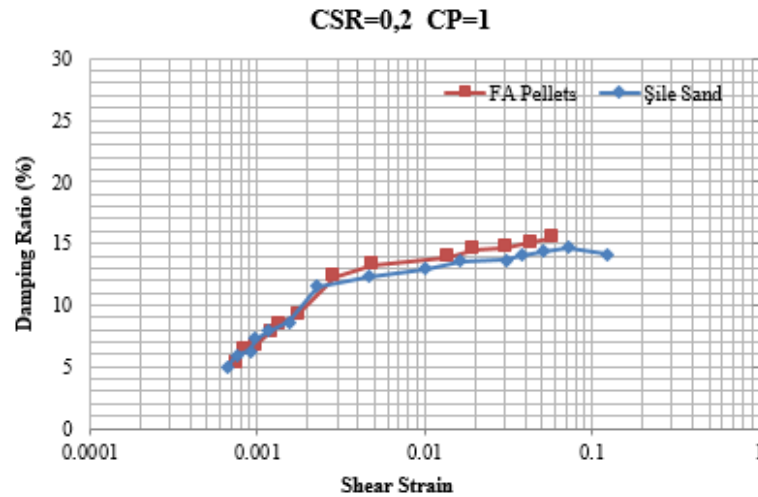


Figure 4.25. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions ($CP=1$ $CSR=0.2$).

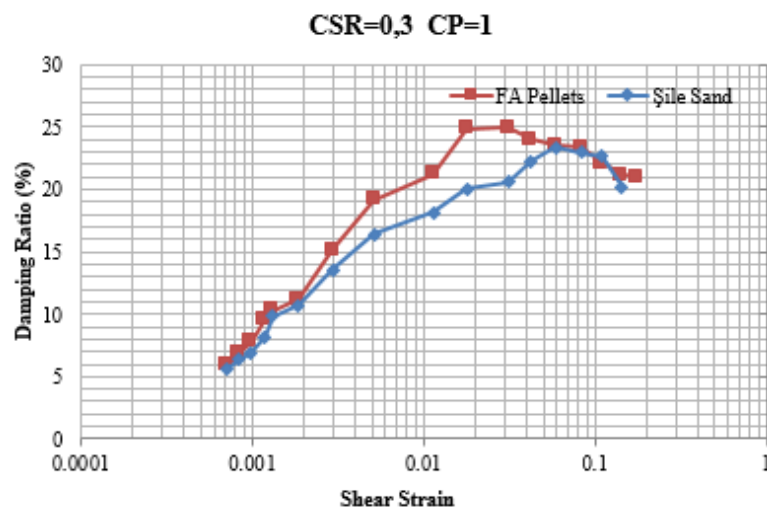


Figure 4.26. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions ($CP=1$ $CSR=0.3$).

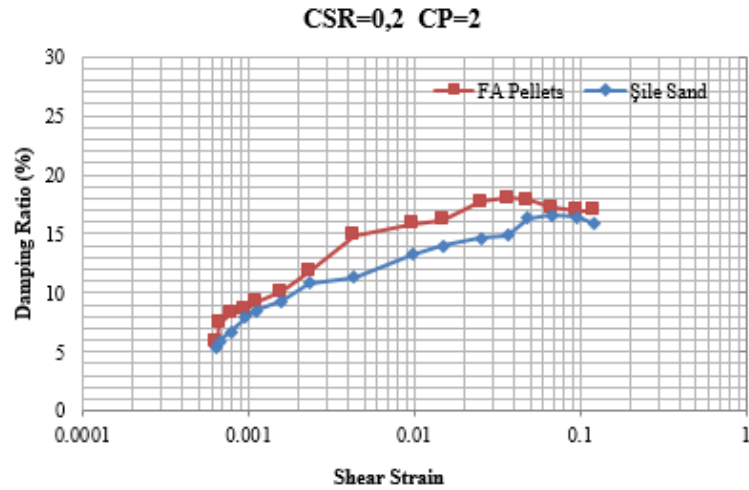


Figure 4.27. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2).

4.5.2.2. Kilyos Sand and the Fly Ash Pellets with Same Gradation. The damping curves of Kilyos Sand and the poorly graded fly ash samples are given in the Figure 4.29, under CP=1 sec and CSR=0.2 loading condition. The graph shows a typical behavior for sands, not differing a lot from 50 kPa consolidation. The damping ratio values of fly ash pellets are considerably higher before 0.4% strain rate. As seen in the figure the pellets are losing the good behavior after that threshold and start to have smaller damping ratio values. As for sand, after the same strain range, increase of damping ratio slows and stabilizes within a range and increase slightly at the end. This trend is also present at the same gradation fly ash pellet sample as given in the figure 4.16.

Although the two different sets of damping curves shows the same trend of increase and a decrease after a threshold, the average of damping ratio values tend to be slightly smaller for fly ash pellets with the same gradation for CP=1 and CSR=0.2 loading condition.

The effect of cyclic stress ratio is also visible seen the Figure 4.29. The damping ratio values are significantly higher when the material is tested under higher CSR values. This increase is much more visible at fly ash pellet specimens. The fly ash pellet sample is not experiencing a sharp decrease as it experienced in smaller cyclic

stress ratio value. The behavior of sand is nearly not affected by the increase of CSR in terms of the general trend.

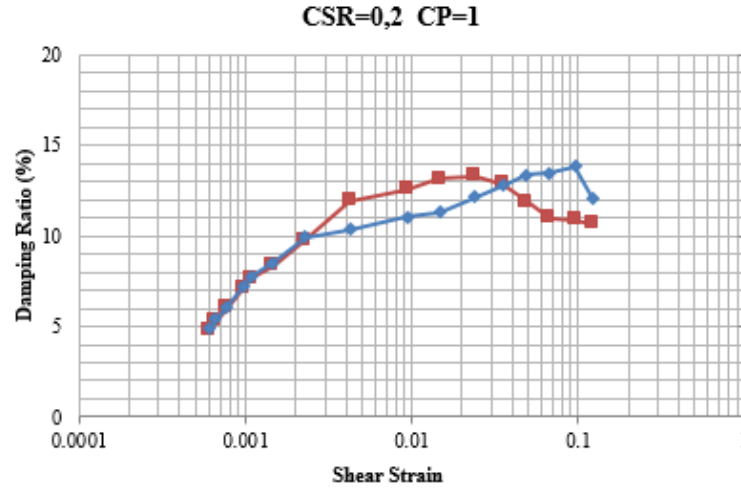


Figure 4.28. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.2).

The same trend of increase of damping ratio as the strains are increasing is noted also for the higher loading condition for both Kilyos Sand and fly ash pellets. As seen in the Figure 4.18, under cyclic stress ratio of 0.3 with cyclic period of 1 sec, fly ash pellets exhibit a higher damping value. This shows that fly ash pellets dissipate bigger loads than same sized sand samples.

After the strain levels of 0.2%, the trend for fly ash pellets and sand particles differ remarkably. The fly ash pellets exhibit a remarkable increase around 0.2% of shear strain, and this increase trend stays valid till failure. Same increase is observed for Kilyos Sand but the increase of damping ratio of fly ash pellets is steeper than the sand. At higher CSR values, it is seen that the damping ratio values are remarkably higher, especially for fly ash pellets.

The same effect can be also seen at the third graph. As the cyclic period is doubled, damping ratio values increased slightly too. This increase is not as much as when the cyclic stress ratio increase but still a slight increase is present. But as in the first graph, the fly ash pellets experience a decline around 0.4% shear strain values.

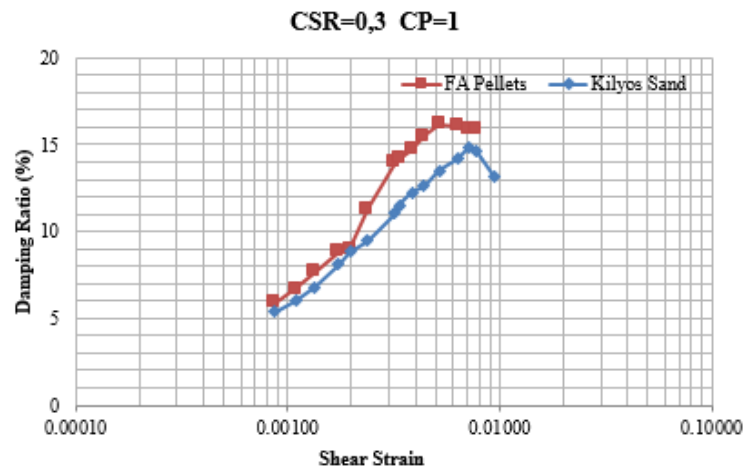


Figure 4.29. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=1 CSR=0.3).

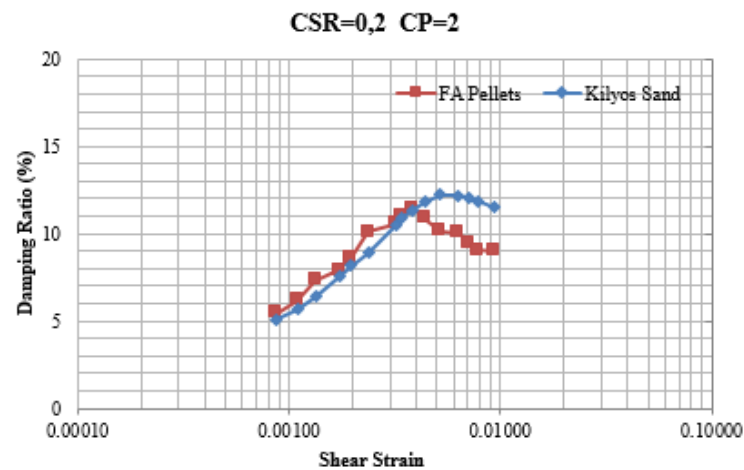


Figure 4.30. Shear Modulus Reduction Curves of Kilyos Sand and the Same Graded Pellets at given Loading Conditions (CP=2 CSR=0.2).

4.5.3. The Variation of Damping Ratios Versus Relative Density

In this part, the variation of damping ratio versus relative density will be studied. As it is seen in the Figure 4.32, both the damping ratio values of Sile sand and well graded fly ash pellet experience higher energy dissipation for higher relative density values. But it is clearly seen that even for high relative density, which represent loosest state of soil, fly ash pellets provide a remarkable high damping ratios. As for the relative density=0.90, damping ratio values are exceeding 13% for well graded fly ash

pellets, which is at the same range of 0.57 relative density till 1% of shear strain. It is seen that the fly ash pellets have certainly good damping characteristics even in loose state compared to other type of materials.

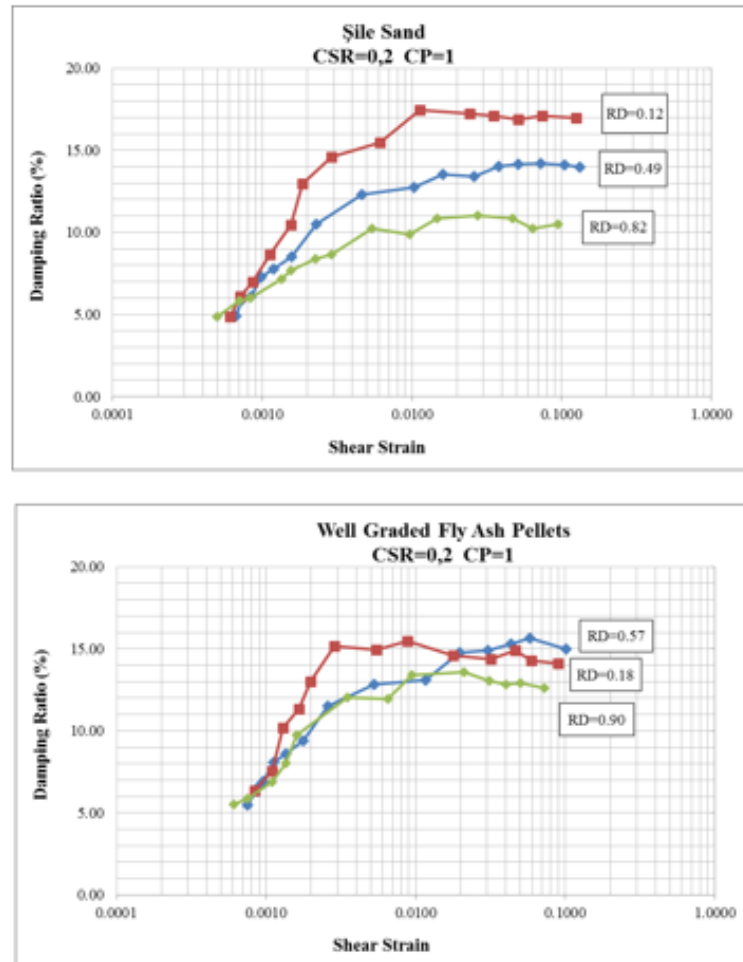


Figure 4.31. The Effect of Relative Density on Damping Ratio for Sile Sand and Well Graded Fly Ash Pellets.

For instance, for Sile Sand, even though the damping ratio values tend to be close at the smallest strain values, the effect of relative density is clearly present throughout all the curves. The loose state, which is the state with highest relative density, shows a poor damping ratio values, whereas the dense state, which is the state with lowest relative density, exhibits much higher damping ratio values.

Furthermore, a similar behavior can be observed for Kilyos sand and poorly graded fly ash pellets. The effect of changing of relative density for Kilyos sand is

excessively remarked as shown in Figure 4.21. The damping ratio curves exhibit an increasing trend for smaller relative density as it was noted for Sile Sand. So it can be said that in general the damping ratio increase for smaller relative density. In other words; more the sands are compacted, more they dissipate energy. But for fly ash pellets, even for high relative density, which shows the loose state of soil, the damping ratio values are sufficiently high.

In Figure 4.22, the damping ratio values of poorly graded fly ash pellets are given. The same behavior experienced for well graded fly ash pellets are present also for poorly graded sample. The effect of relative density is present also for this sample, but it is not remarkable as Kilyos sand.

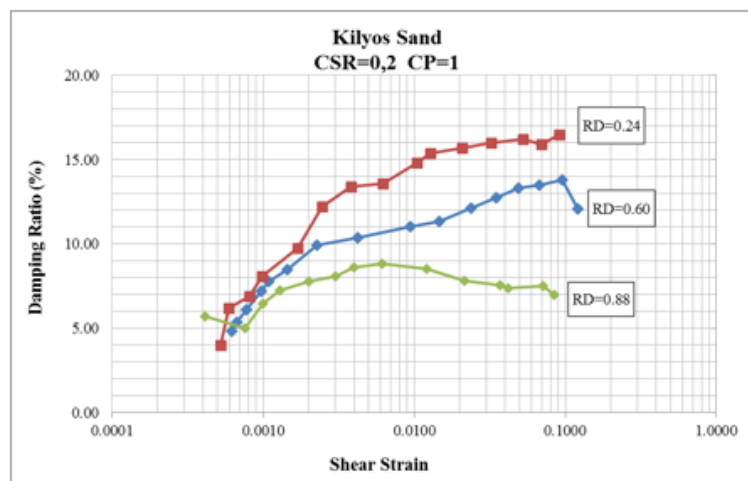


Figure 4.32. The Effect of Relative Density on Damping Ratio for Kilyos Sand under Given Loading Conditions.

Another remark is that, as for sand sample, the final damping ratio value at 10% shear strain is 17, 12 and 7% respectively for 0.24, 0.60, and 0.88 relative density values, whereas they are about 12.5, 11.5, 11% respectively for 0.21, 0.85, 0.54 relative density values. As the increasing relative density differ 10% damping ratio for sand, the effect of change same rate of relative density for fly ash pellets is as few as 1.5%. This exorbitant difference data for same graded sand and fly ash pellets show that for this loading conditions, the fly ash shows an extremely better dissipating characteristic when the conditions are not available for compaction.

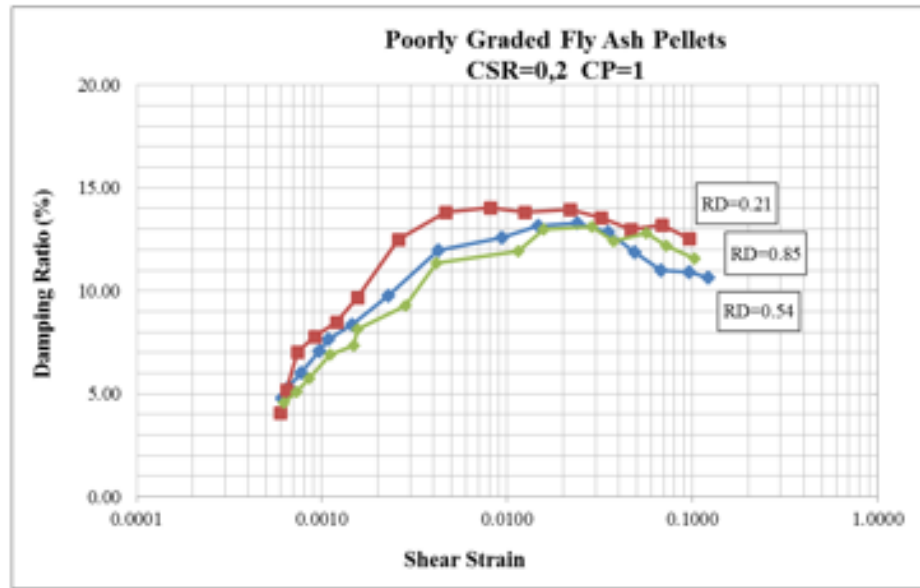


Figure 4.33. The Effect of Relative Density on Damping Ratio for Kilyos Sand and Poorly Graded Fly Ash Pellets.

4.5.4. The Variation of Number of Cycles to Arrive 0.1 Strain Level

As it is noted in the methodology, the strain levels that are generally encountered in geotechnical engineering are known. By referring this knowledge, 0.1 strain level is selected as a threshold to identify the characteristics of materials under cyclic loading. Under 50 kPa consolidation pressure the graphs are plotted and the relations are established in Figure 4.36 and 4.37. It is clearly seen that number of cycles to arrive 0.1 shear strain value is the highest for Sile Sand. For Sile Sand number of cycles to arrive 0.1 shear strain is ranging around 16-21 cycles as for Kilyos sand this value is around 14-18 cycle.

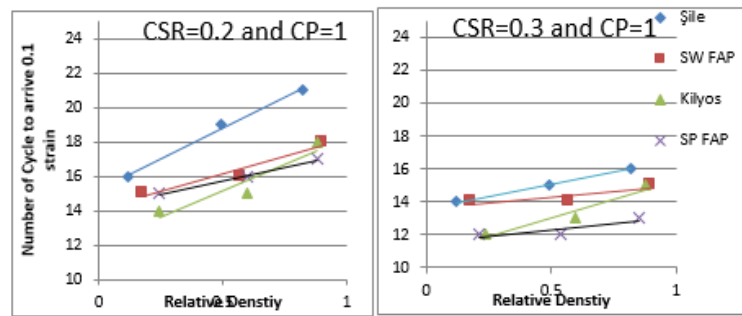


Figure 4.34. The Number of Cycles to Arrive 0.1 Strain.

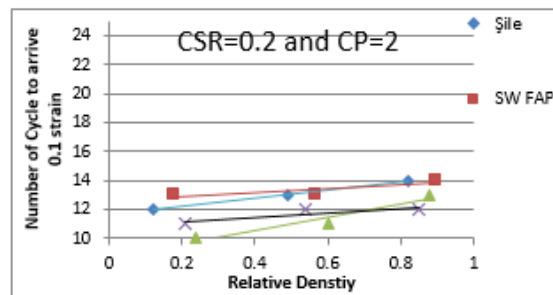


Figure 4.35. The Number of Cycles to Arrive 0.1 Strain.

Under 100 kPa consolidation pressure the similar graphs are plotted and the relations are established in Figure 4.38 and 4.39. It is clearly seen that number of cycles to arrive 0.1 shear strain value is higher for Sile Sand. For Sile Sand number of cycles to arrive 0.1 shear strain is ranging around 15-19 cycles as for Kilyos sand this value is around 13-17 cycle. The same difference can be said also for different graded fly ash pellets but less remarkably. And it is also noted that especially for poorly graded fly ash pellets, the effect of increase of relative density is stayed limited.

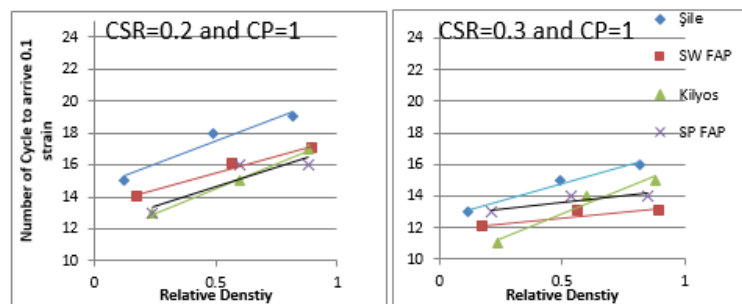


Figure 4.36. The Number of Cycles to Arrive 0.1 Strain.

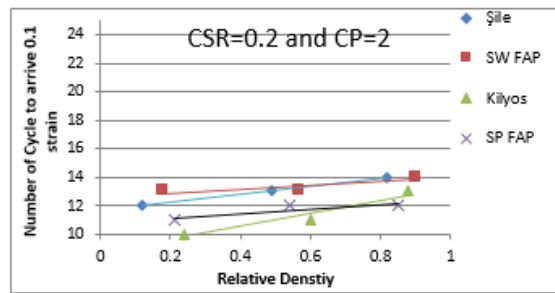


Figure 4.37. The Number of Cycles to Arrive 0.1 Strain.

4.5.5. G_{sec} values corresponding different strain values

As the degradation of shear modulus curves give, by increasing shear strain, the shear modulus values decrease. In order to see this reduction clearer, some bar charts are presented in this chapter.

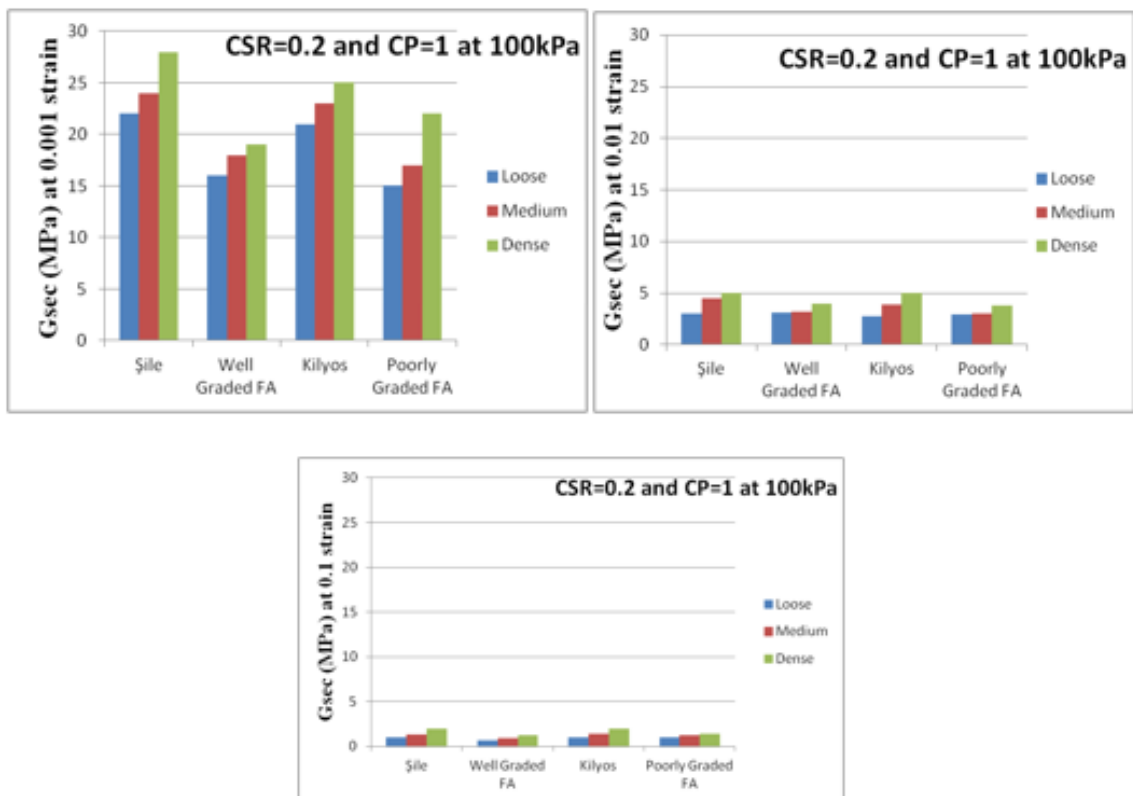


Figure 4.38. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=1 Loading Conditions.

Figure 4.38 shows the variation of secant shear modulus values at 0.001 strain for 4 different materials that are used in this study in 3 different relative density. The clear trend of increase in shear moduli can be seen as the specimen gets denser. This makes coherence with the results given in the literature for various sands. For fly ash pellets, the increase is also present but not as remarkable as for sands. Even though, it can be argued that the fly ash pellets gets stiffer as they are compacted.

Furthermore, the same trend is observed for larger strain level. At 0.01 strain, a remarkable increase of shear modulus values, as the specimen gets denser. For Sile Sand, the shear modulus is at 3 MPa for loose condition. As the same sand is tested after compaction, it gives much stiffer behavior with shear modulus value of 5 MPa.

As it was stated for 0.01 shear strain, the same trend is seen for fly ash pellets but less remarkably for $CSR=0.2$ and $CP=1$ loading conditions. Especially for poorly graded fly ash pellets, where the loose sample gave nearly 3 MPa and the dense sample nearly 4 MPa. The increase stayed at smaller rates. For well graded fly ash pellets, the increase is seen clearer. The loose specimen has a shear modulus of 3 MPa, as for dense specimen it just increased to 5 MPa. For loading condition of $CSR=0.3$ and $CP=1$ as it is given in the Figure 4.40, similar behavior can be observed. There is a clear increase of shear modulus values for all the specimen as they get denser which means higher relative density values.

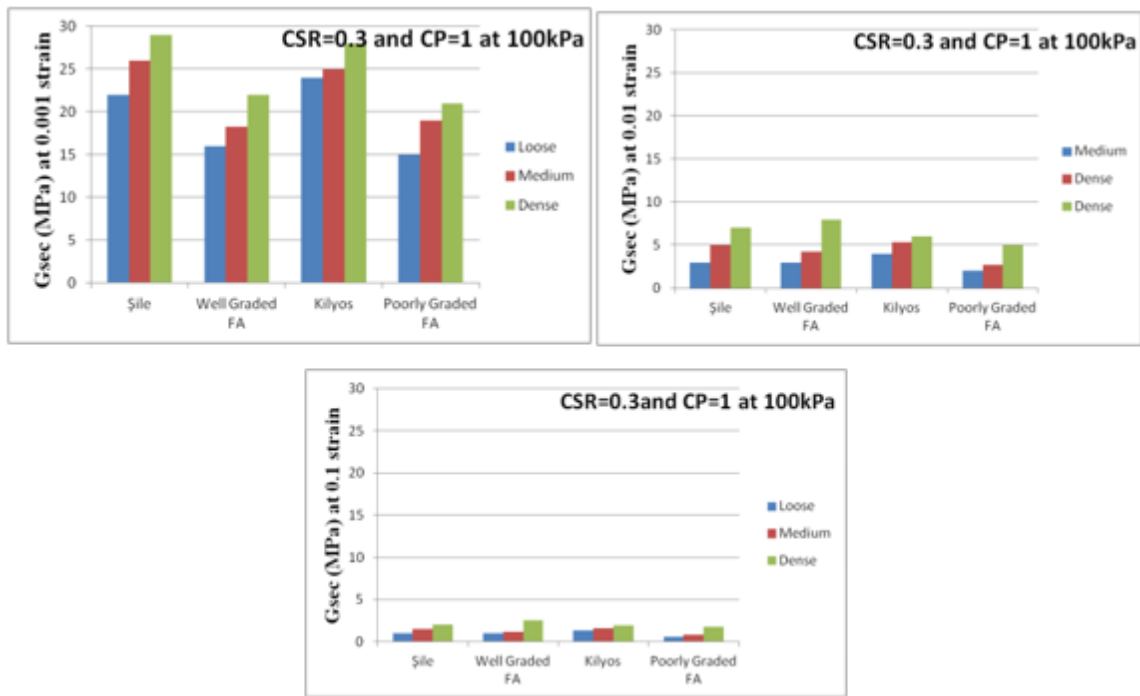


Figure 4.39. Shear Moduli at Different Shear Strain Rate for CSR=0.3 and CP=1 Loading Conditions.

Moreover, the same trend of increase is still present for larger strain level. The shear moduli of 4 different specimens at 3 different relative densities are given for the 0.1 strain rate in Figure 4.40c. At 0.1 strain value, there is a remarkable increase of shear modulus values, as the specimen gets denser. And also the shear modulus values are degraded heavily. As the shear modulus values for well graded fly ash pellets is around 20 MPa at 0.001 strain value, at 0.1 strain it degraded as much as that it arrive to 1.2 MPa. For Şile Sand, the shear modulus is at 1 MPa for loose condition at 0.1 strain values. As the same sand is tested after compaction, it gives much stiffer behavior with shear modulus value of 2 MPa.

As it was stated for 0.01 shear strain, the same trend is seen for fly ash pellets but less remarkably for CSR=0.2 and CP=1 loading conditions. Especially for poorly graded fly ash pellets, where the loose sample gave 1 MPa and the dense sample 1.40 MPa. The increase stayed at smaller rates. For well graded fly ash pellets, the increase is seen clearer. The loose specimen has a shear modulus of 0.7 MPa, as for dense

specimen it just increased to 1.2 MPa. For loading condition of $CSR=0.2$ and $CP=2$ as it is given in the Figure 4.40, similar behavior can be observed. There is a clear increase of shear modulus values for all the specimen as they get denser which means higher relative density values.

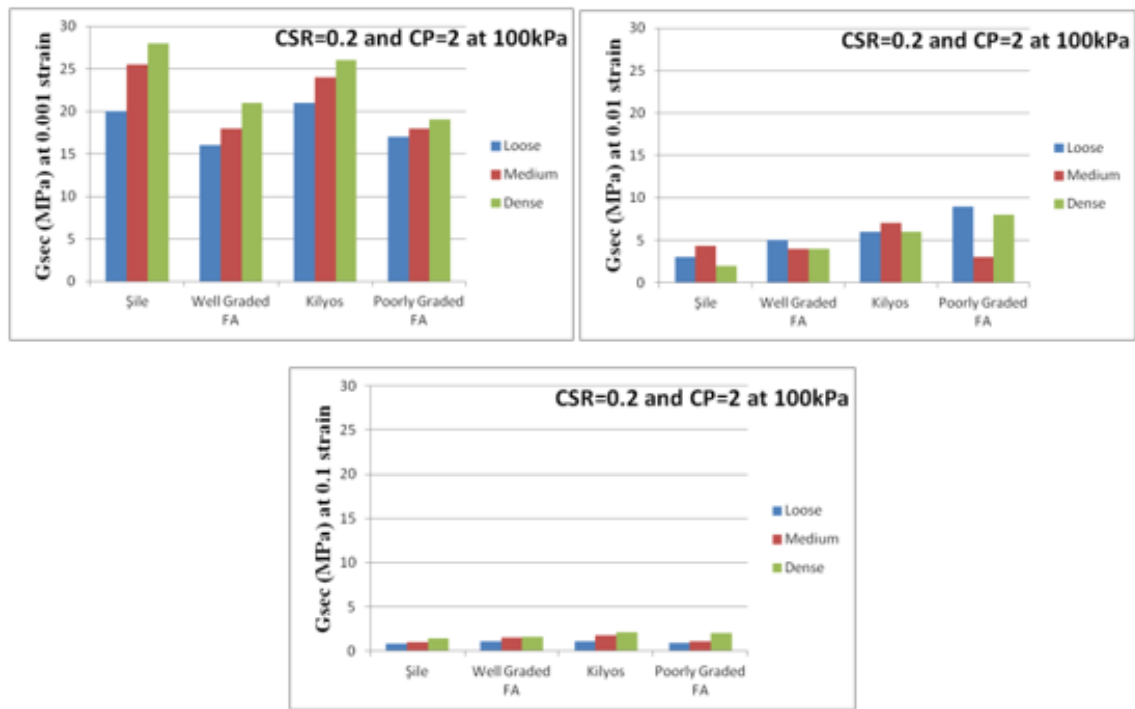


Figure 4.40. Shear Moduli at Different Shear Strain Rate for $CSR=0.2$ and $CP=2$ Loading Conditions.

The same graphs are also obtained for 50 kPa consolidation pressure and given below. The Figure 4.40 and 4.41 shows the variation of secant shear modulus values at 0.001 strain for 4 different materials that are used in this study in 3 different relative density. The clear trend of increase in shear moduli can be seen as the specimen gets denser. This makes coherence with the results given in the literature for various sands. For fly ash pellets, the increase is also present but not as remarkable as for sands. Even though, it can be argued that the fly ash pellets gets stiffer as they are compacted.

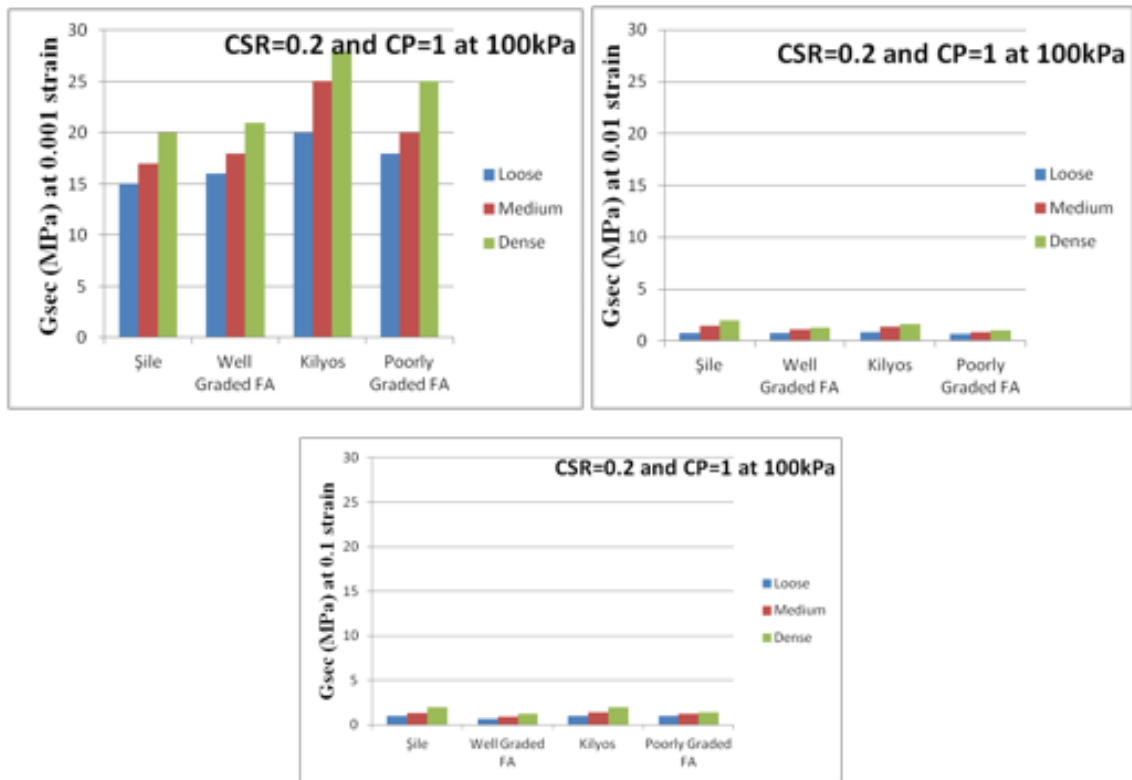


Figure 4.41. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=1 Loading Conditions.

The similar behavior of increase is also present for larger strain level. The shear moduli of 4 different specimens at 3 different relative densities are given for the 0.01 strain rate in Figure 4.43. At 0.1 strain value, the clear increase of shear modulus values is seen, as the specimen gets denser. And also it has to be noted that the shear modulus values are degraded heavily. As the shear modulus values for well graded fly ash pellets is around 20 MPa at 0.001 strain value, at 0.1 strain it degraded as much as that it arrive to 1.2 MPa. For Şile Sand, the shear modulus is at 1 MPa for loose condition. As the same sand is tested after compaction, it gives much stiffer behavior with shear modulus value of 2 MPa.

As it was stated for 0.01 shear strain, the same trend is seen for fly ash pellets but less remarkably for CSR=0.2 and CP=1 loading conditions. Especially for poorly graded fly ash pellets, where the loose sample gave 1 MPa and the dense sample 1.40 MPa. The increase stayed at smaller rates. For well graded fly ash pellets, the increase

is seen clearer. The loose specimen has a shear modulus of 0.7 MPa, as for dense specimen it just increased to 1.2 MPa. For loading condition of $CSR=0.2$ and $CP=2$ as it is given in the Figure 4.43, similar behavior can be observed. There is a clear increase of shear modulus values for all the specimen as they get denser which means higher relative density values.

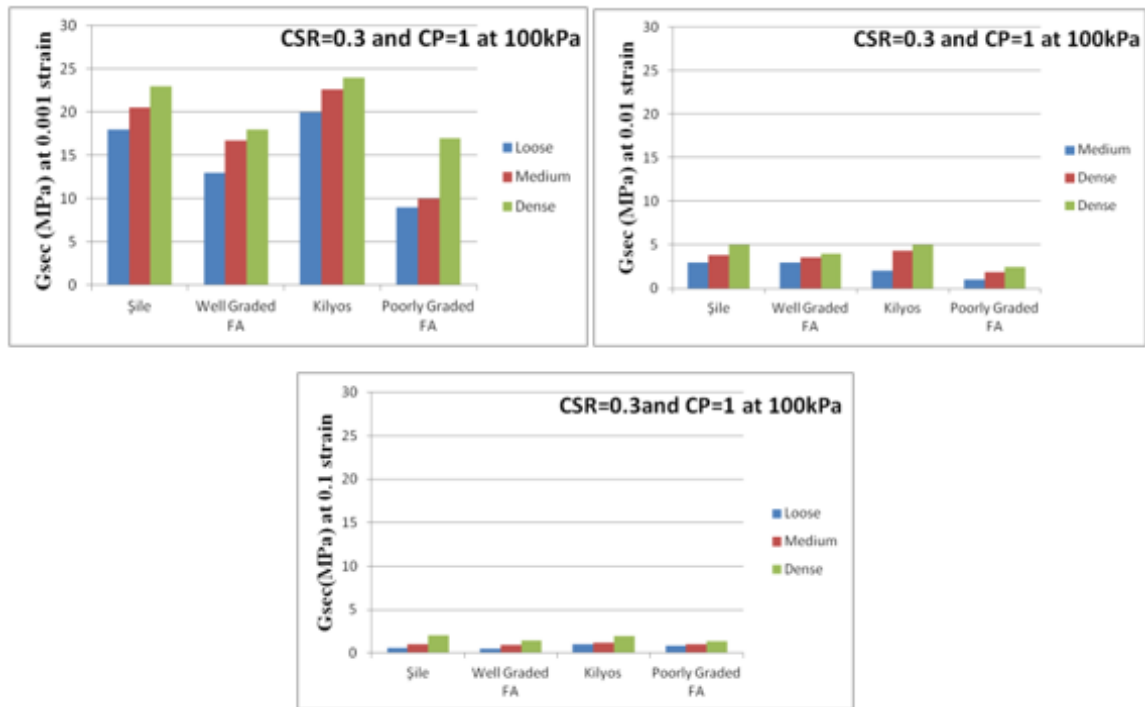


Figure 4.42. Shear Moduli at Different Shear Strain Rate for $CSR=0.3$ and $CP=1$ Loading Conditions.

The similar behavior of increase is also present for larger strain level. The shear moduli of 4 different specimens at 3 different relative densities are given for the 0.01 strain rate in Figure 4.42. At 0.1 strain value, the clear increase of shear modulus values is seen, as the specimen gets denser. And also it has to be noted that the shear modulus values are degraded heavily. As the shear modulus values for well graded fly ash pellets is around 20 MPa at 0.001 strain value, at 0.1 strain it degraded as much as that it arrive to 1.2 MPa. For Şile Sand, the shear modulus is at 1 MPa for loose condition. As the same sand is tested after compaction, it gives much stiffer behavior with shear modulus value of 2 MPa.

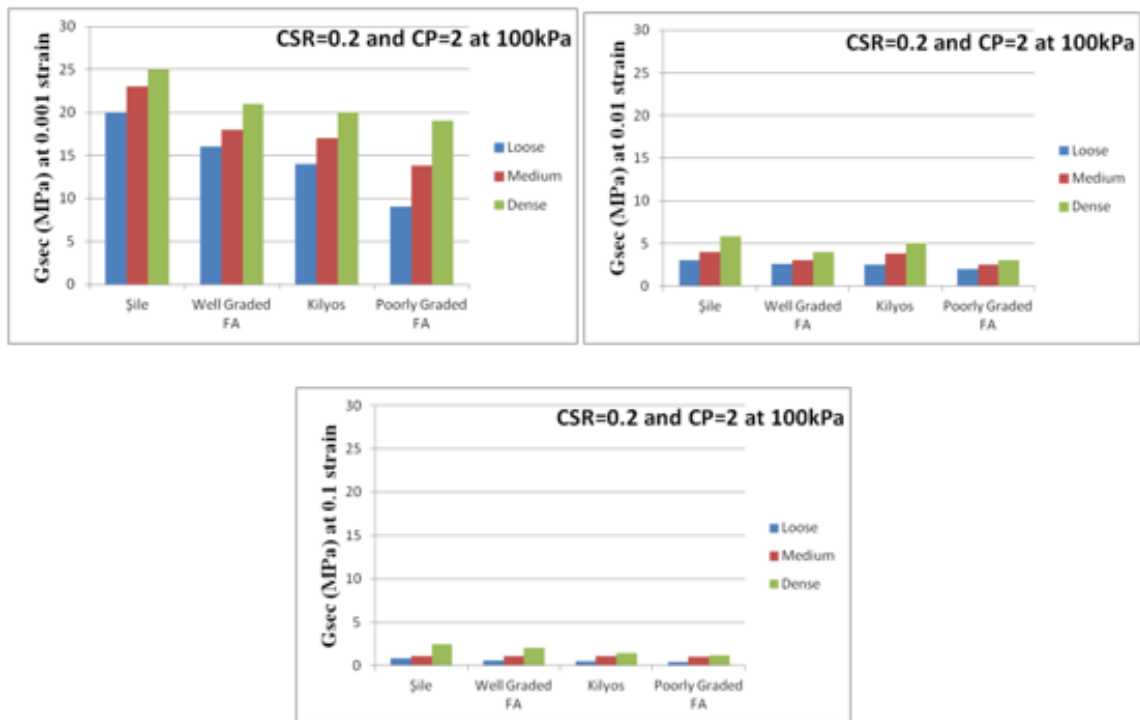


Figure 4.43. Shear Moduli at Different Shear Strain Rate for CSR=0.2 and CP=2 Loading Conditions.

As it was stated for 0.01 shear strain, the same trend is seen for fly ash pellets but less remarkably for CSR=0.2 and CP=1 loading conditions. Especially for poorly graded fly ash pellets, where the loose sample gave 1 MPa and the dense sample 1.40 MPa. The increase stayed at smaller rates. For well graded fly ash pellets, the increase is seen clearer. The loose specimen has a shear modulus of 0.7 MPa, as for dense specimen it just increased to 1.2 MPa. For loading condition of CSR=0.2 and CP=2 as it is given in the Figure 4.43, similar behavior can be observed. There is a clear increase of shear modulus values for all the specimen as they get denser which means higher relative density values.

4.5.6. The Effect of Void Ratio to the Shear Modulus Values

In this chapter, the change of stiffness, secant shear modulus values are evaluated for different void ratios. As it is seen in the Figure 4.45 and Figure 4.46, the void ratio values for fly ash pellet specimens are much higher than the same graded sand samples.

But it is also seen that the shear modulus values are found to be in the same range. For 100 kPa consolidation pressure, for 0.1 shear strain value, it is clearly seen that the well graded fly ash pellets has better stiffness values with respect to Sile Sand which has the same gradation. The same can be said for the poorly graded fly ash pellets, when they are compared to Kilyos Sand, despite to their high void ratio, they stay at the same range and for a void ratio around 1.00, they have much better characteristics than the same void ratio Kilyos Sand, obtaining 20 MPa secant modulus.

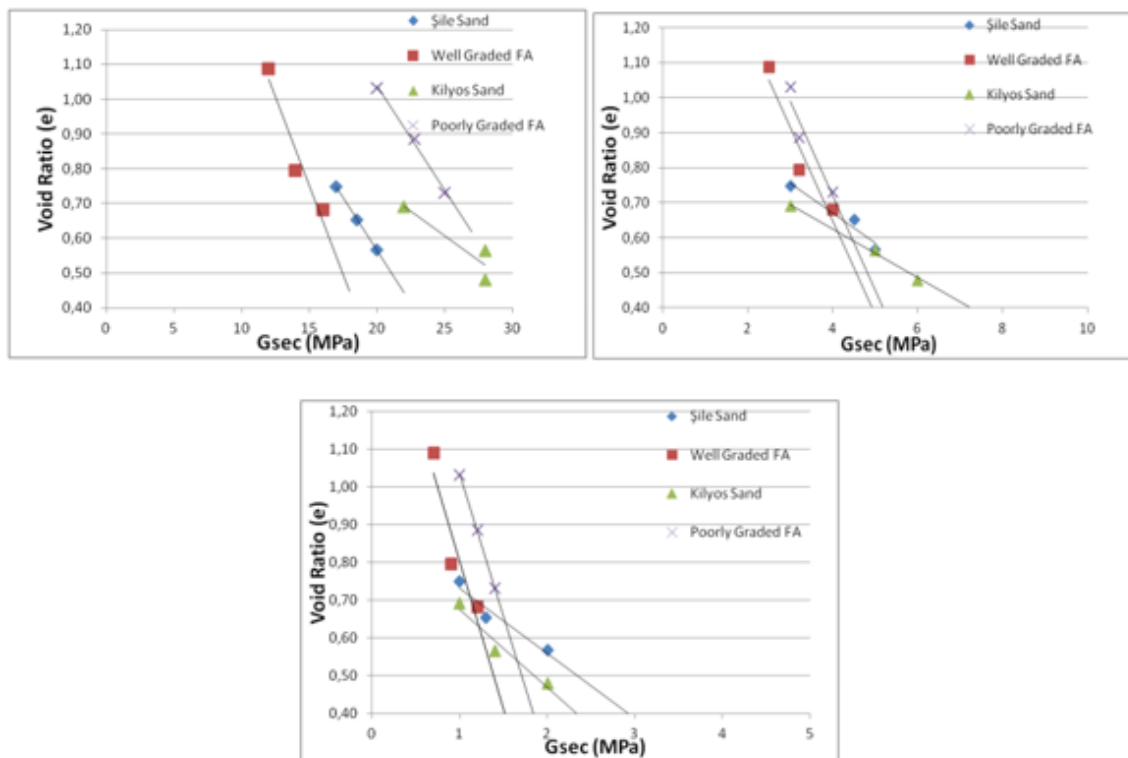


Figure 4.44. Secant Shear Modulus for 0.001, 0.01 and 0.1 Shear Values at Different Void Ratio Values under 100 kPa Consolidation Pressure.

The same characteristics are visible for 50 KPa consolidation pressure. Despite their high void ratio, they achieve the same range stiffness values of the sand samples. Another remark is that the different fly ash pellets tend to have nearly the same rate of change of shear modulus versus void ratio. As it can be seen at Figure 4.45 b,c and Figure 4.46 b,c the fitted line for fly ash samples has nearly the same angle. This characteristic cannot be seen for the sand samples. So it can be said that the fly ash samples has similar compaction characteristics.

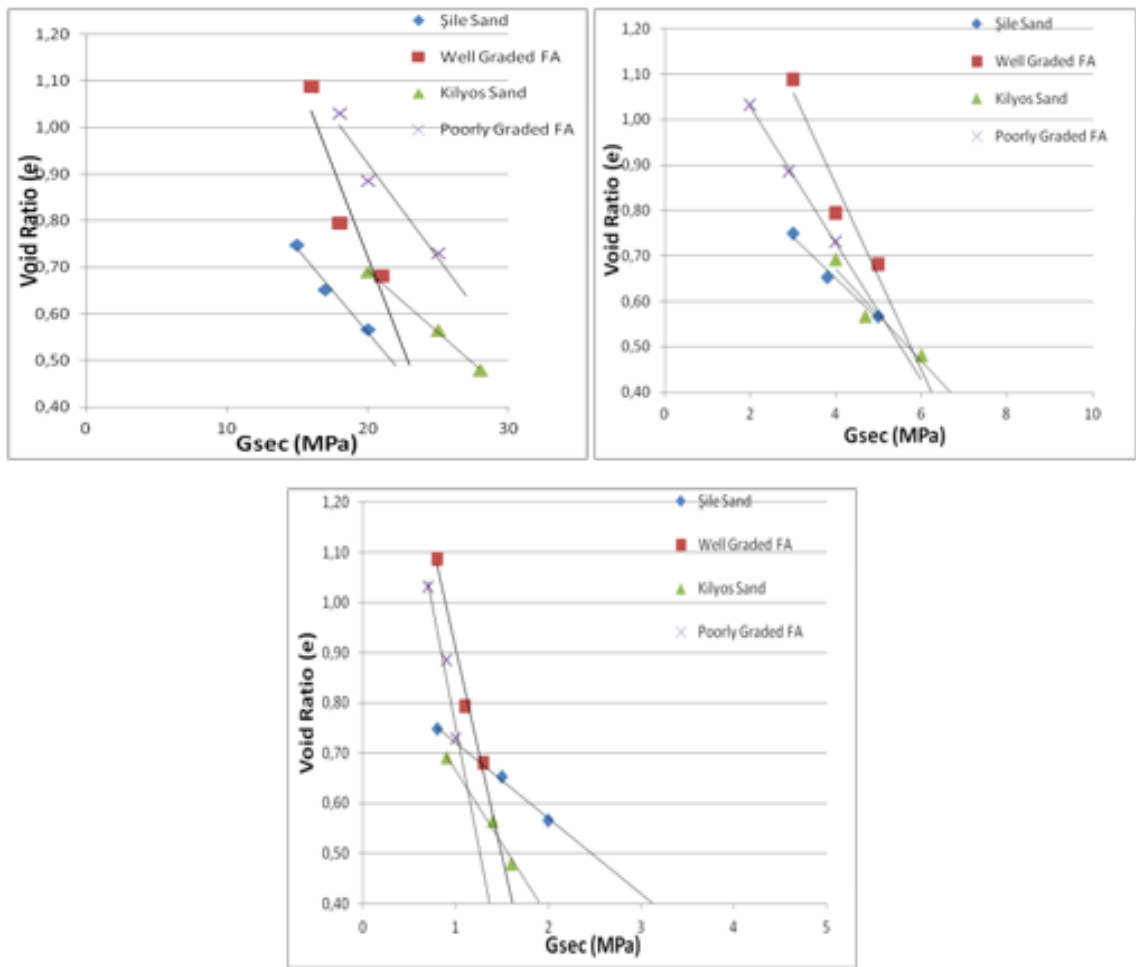


Figure 4.45. Secant Shear Modulus for 0.001, 0.01 and 0.1 Shear Values at Different Void Ratio Values under 50 kPa Consolidation Pressure.

5. SUMMARY

Monotonic triaxial and cyclic simple shear tests results have been presented for two different gradation of pelletized fly ash, as well as for two different sands. The objective was to obtain the cyclic behavior parameters of manufactured sand (fly ash pellets) and then to compare them to quartz sands.

For the monotonic behavior, the fly ash pellets have slightly smaller friction angles. The range of internal friction angles of sands studied was between 36° - 40° . For the manufactured sand the range was between 31° - 38° .

In general, the fly ash pellets has greater void ratios than sands and this lead them to have smaller unit weight values as well as crushing of their particles. So a crushing behavior was expected during cyclic loading.

Under cyclic loading, fly ash pellets behave in the same trend as Kilyos and Sile sands. Their secant shear modulus values degraded with shear strain following the same trend under same loading conditions. The same trends observed also for damping ratio values and the dependencies to some parameters such as the effect of normal and cyclic loading conditions, relative density, are also found in the same trends. But even though these general similarities are noted, there are some remarkable differences in the behavior as well.

The relative density does not have a great influence on the damping behavior of fly ash pellets as much as on the sand samples. As compacting sand samples results up to 5% of damping ratio change, for fly ash pellets it differs less than 2%. This means they dissipate well the energy even in the loose condition and it has satisfactory damping characteristics even without any compaction. This behavior can be attributed to the crushing behavior of fly ash pellets. Even though the crushing rate stayed at certain levels, it contributes material to dissipate more energy. The secant shear modulus values are

The fly ash pellets are found not crushed harshly under cyclic loads as expected. As it is shown by the crushing test results, they have enough resistance under direct point loading, also for the cyclic loading the crushing of particles remained less than 5% for 50 kPa loading condition, and always remained under 12% of the samples under 100 kPa consolidation stress.

Stiffness and damping ratio are found dependent on CSR. For both fly ash pellets and sand samples, damping ratio decreases as the CSR increasing. That means fly ash pellets dissipate less energy, for the large cyclic stresses. But still, their damping properties stay at higher values than those of sand. Average stiffness and damping ratio trend curves are lower for the fly ash pellets in comparison to the sand sample. Among the fly ash pellets, stiffness values are nearly the same for the two types of sands and for the range of densities investigated. On the other hand, the poor graded fly ash pellets with significantly more fines, reveals substantially less damping than the sand at shear strains above 0.2%.

So it can be concluded that the fly ash pellets has strength at the same range of the sand of the same gradation. Moreover when the cyclic characteristics are studied, it is found that the fly ash pellets behave stiffer (higher secant modulus) but dissipate more energy during cyclic shear under certain loading conditions (higher damping ratio).

Besides, thanks to its flexibility to produce and its low cost as well as their crushing characteristic and high void ratios, the fly ash pellets t can be used as a reference testing material for study of calcareous sands.

According to Rowe theory and its improved version for soils having crushable grains, some part of applied energy to the sample during the test is consumed to breakage of sand particles. Consumed energy to breakage of grains is a function of both grains crushing strength and crushing amount. Decreasing the relative density and weakening the sand grains, increases the ratio of consumed energy to breakage of particles.

Energy dissipation results as damping in sand. The elastic energy stored in soil element is consumed by the destruction of soil structures, breakage of soil particles, rotation of soil grains, and transformation into other types of energy, including heat and sound. Vucetic et al. (1998) concluded that soil nonlinearity and soil viscosity were the primary contributors to the area of the cyclic stress strain loop. Within approximately 2.0% cyclic shear strain, the damping ratio increased as the cyclic shear strain amplitude; it decreased, however, when the cyclic shear strain amplitude was larger than 2.0%.

6. CONCLUSIONS

The lower sphericity and roundness of quartz sands particles studied resulted in higher stiffness values when compared to that of fly ash pellets especially under higher consolidation stresses. This behavior can be result of the different interlocking behavior due to different angularity and sphericity. At lower consolidation stress, poorly graded Kilyos sand and poorly graded fly ash pellets have higher G_{sec} values compared to that of well graded samples. At 0.01 strain level, the difference between sand and pellets are much smaller.

The numbers of cycles to reach 0.1 strain level are similar for fly ash pellets and sand except Sile Sand which has the highest number of cycles values. With increasing consolidation stress, a slight decrease in number of cycles is observed for all the samples.

Although the void ratio of fly ash pellets are 30% higher than those of quartz sand studied, secant shear modulus values are in the same range. For the smaller strain values, the range is much. The effect of relative density on the damping ratio values are not significant on fly ash pellets as much as on both Kilyos and Sile Sand samples. The damping ratio is increasing for increasing strain values from 0.07 to 0.14 then it decreases which are similar to the values of the values of Brandes' study.

The cyclic properties of manufactured sands are compatible of the studied quartz sands. At small strain, a slight variation occurs, at larger strains no significant was found and they behaved very similar. The high porosity and low unit weight of manufactured sand is similar to those of calcareous sand. The damping ratio and secant shear modulus of the manufactured sand is found in the same range of calcareous sand. Because of this property the parametric studies can be conducted for calcareous sands under various loading conditions. When the manufactured sand is disposed, the cyclic behavior of the manufactured pellets reveal that they can be disposed in large volumes safely in seismic zones.

APPENDIX A: THE SHEAR SECANT MODULUS AND DAMPING RATIO CURVES FOR ALL THE STUDIES DONE

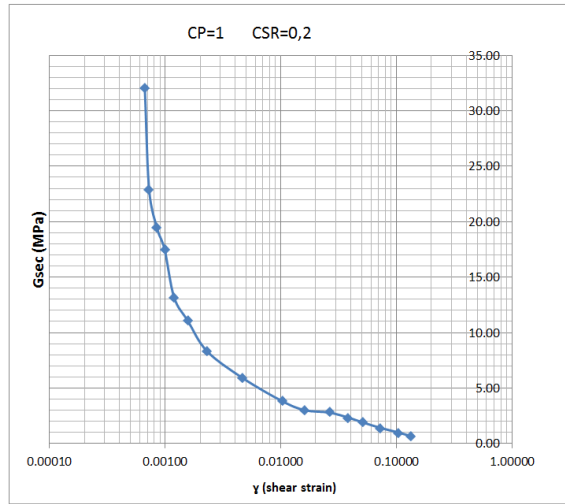


Figure A.1. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

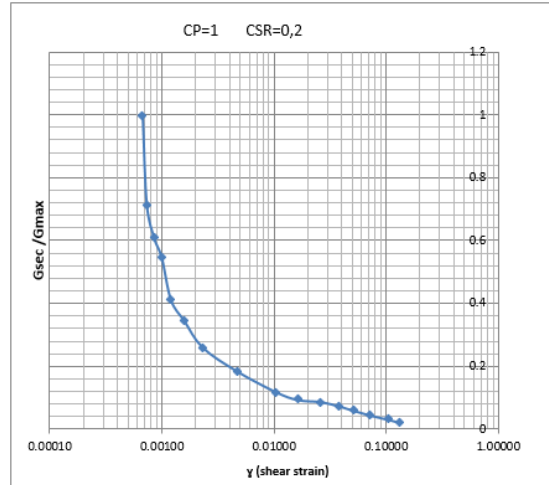


Figure A.2. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

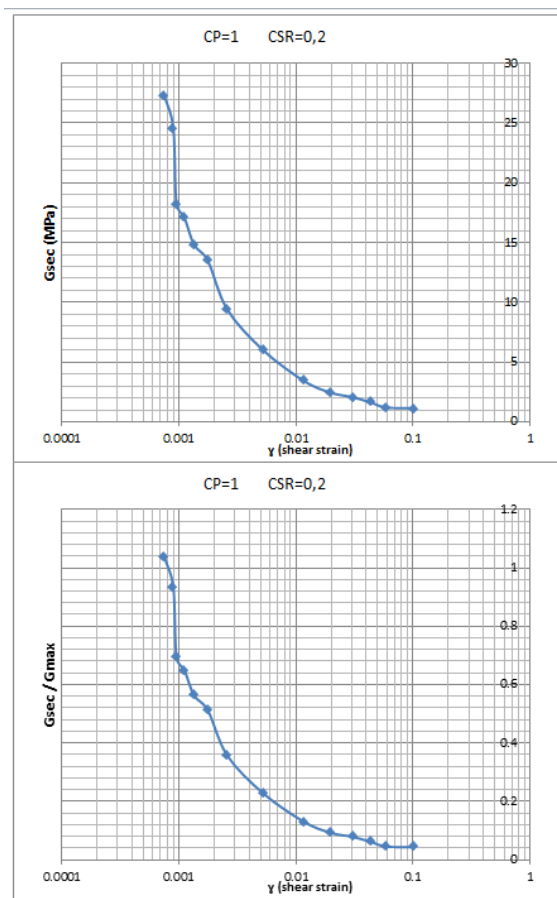


Figure A.3. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions for Well Graded Pellet Sample.

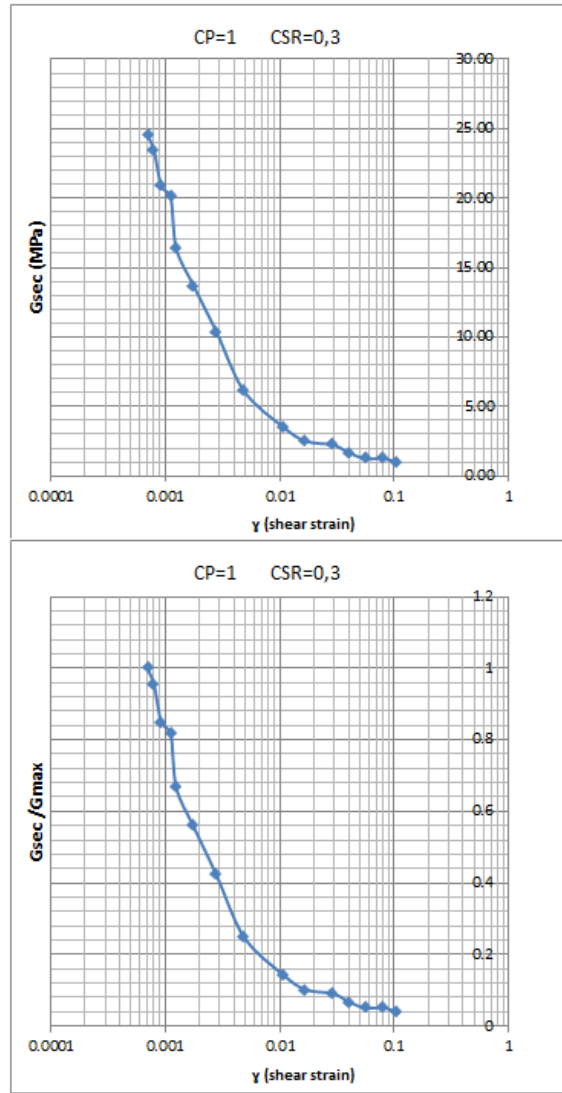


Figure A.4. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.

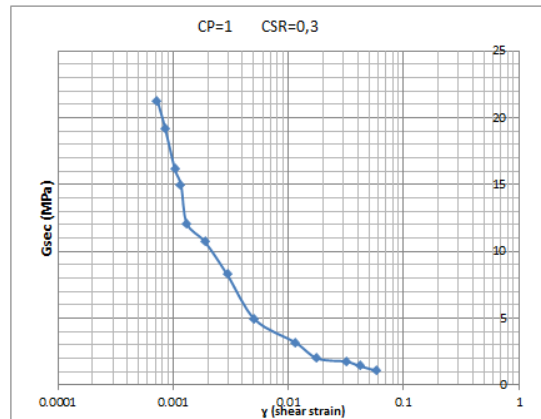


Figure A.5. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.

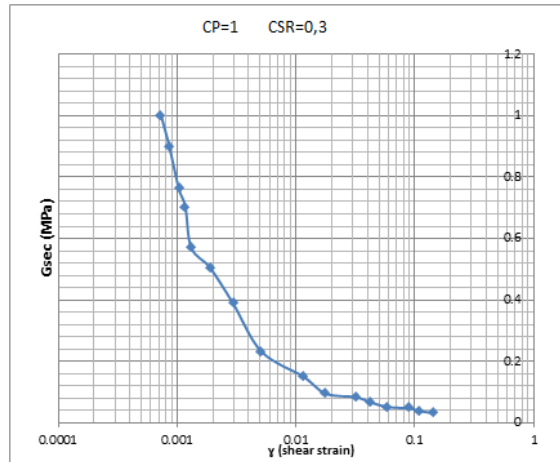


Figure A.6. Normalized Shear Modulus Reduction Curves for well Graded Fly Ash Pellets at Given Loading Conditions.

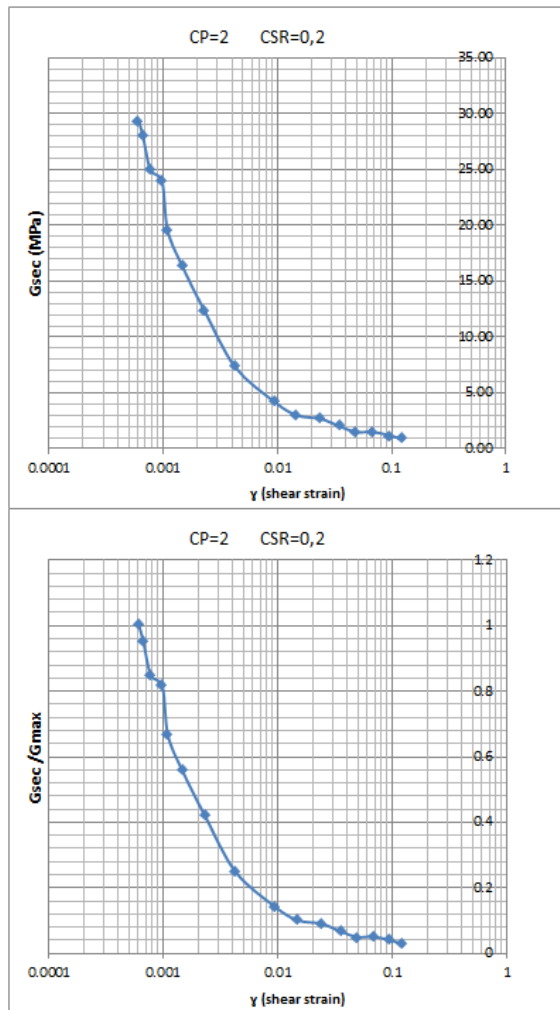


Figure A.7. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.

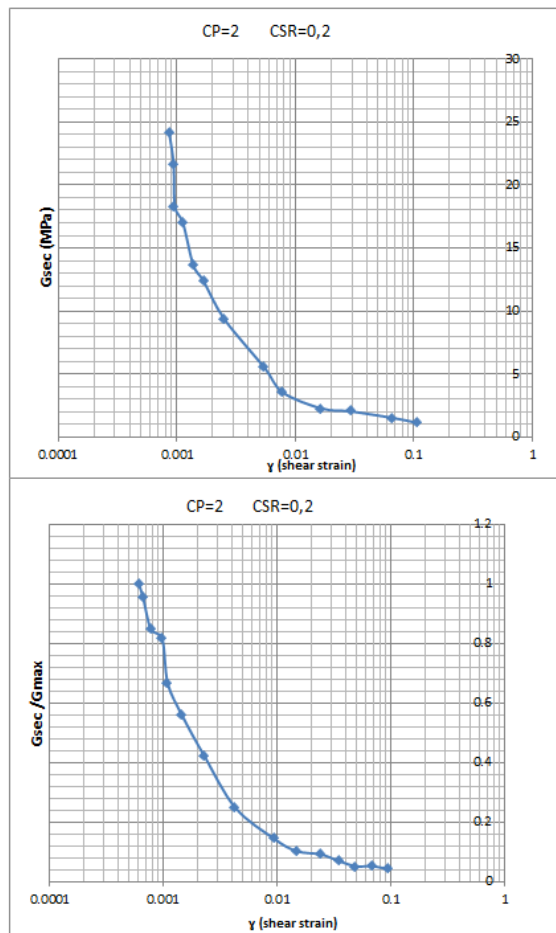


Figure A.8. Shear Modulus Reduction Curves for well Graded Fly Ash Pellets at Given Loading Conditions.

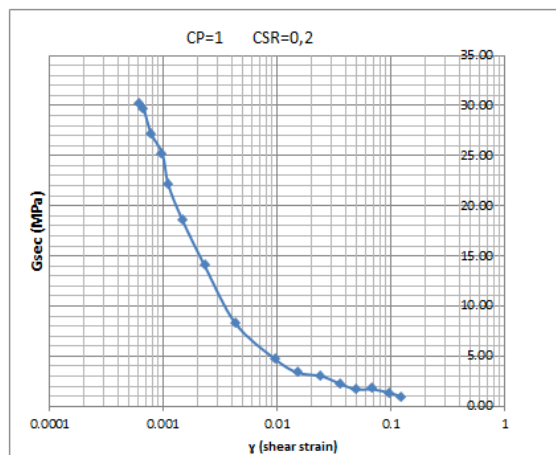


Figure A.9. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

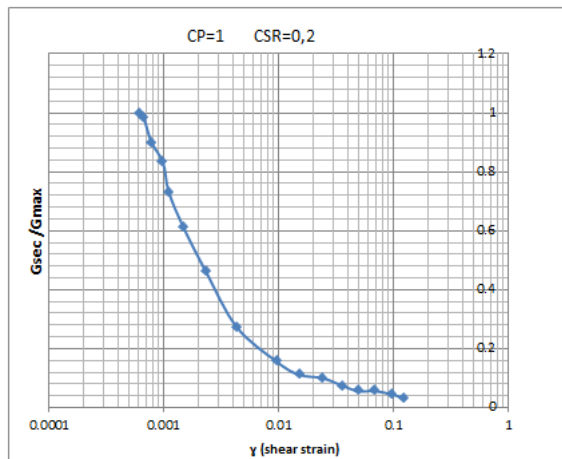


Figure A.10. Normalized Shear Modulus Reduction Curve for Kilyos Sand at Given Loading Conditions.

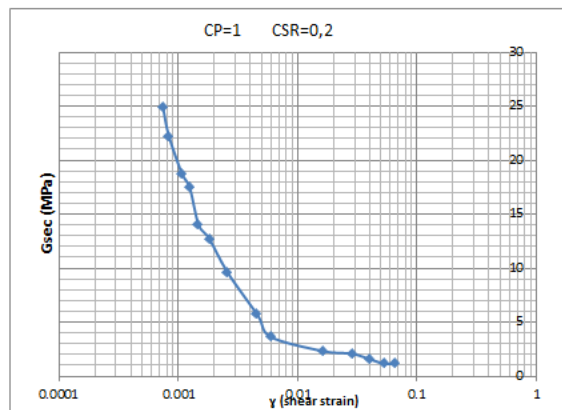


Figure A.11. Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

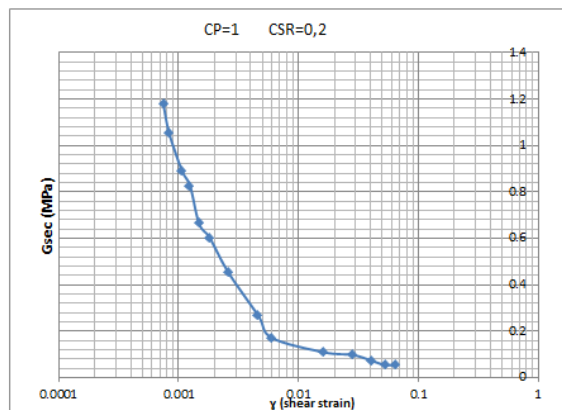


Figure A.12. Normalized Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

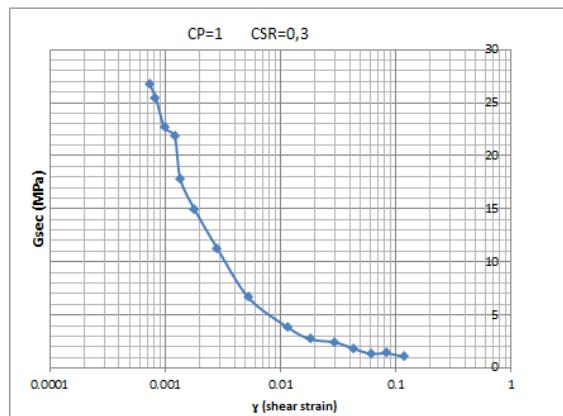


Figure A.13. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

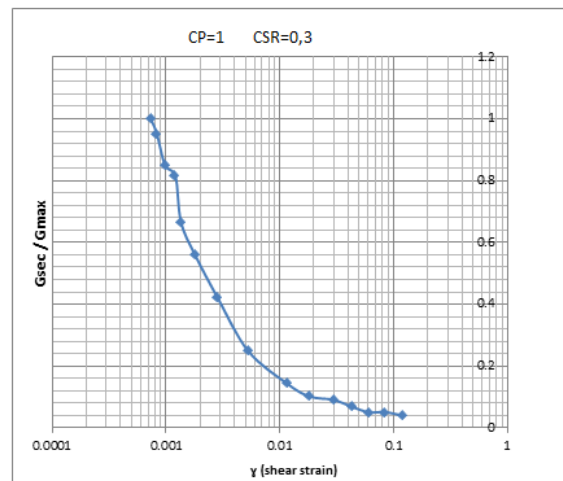


Figure A.14. Normalized Shear Modulus Reduction Curve for Kilyos Sand at Given Loading Conditions.

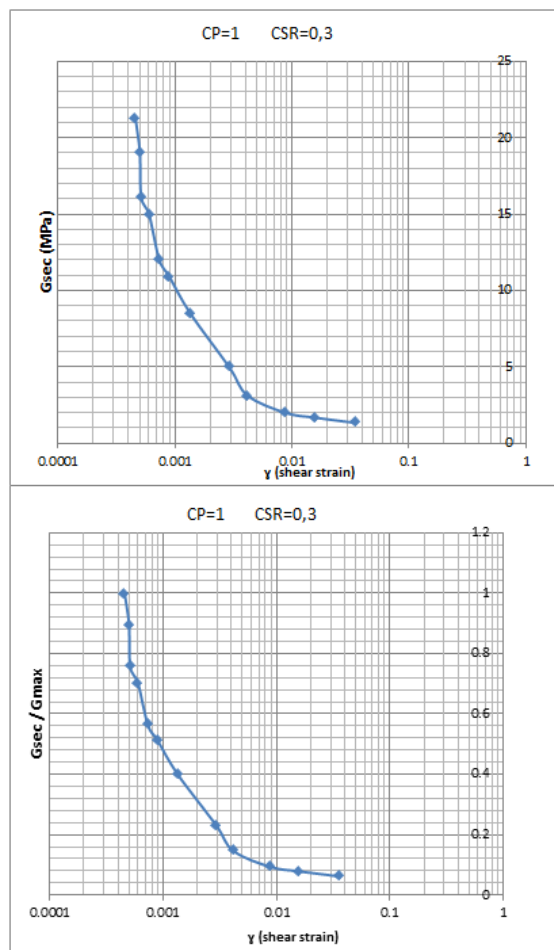


Figure A.15. Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

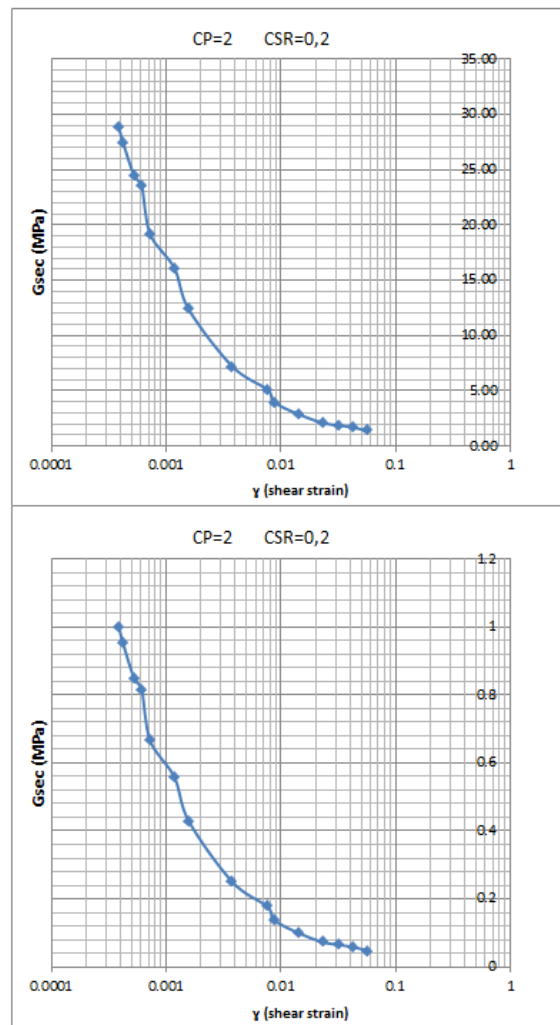


Figure A.16. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

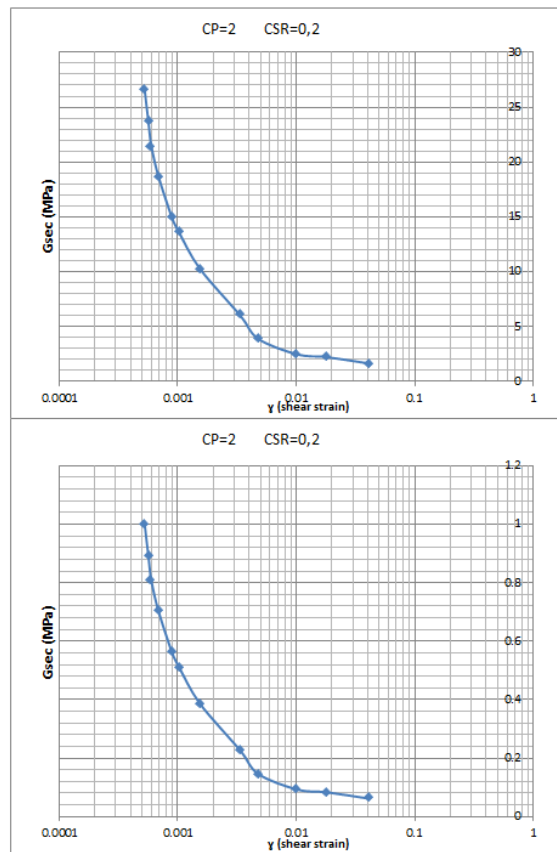


Figure A.17. Shear Modulus Reduction Curves for Poorly Graded at Given Loading Conditions.

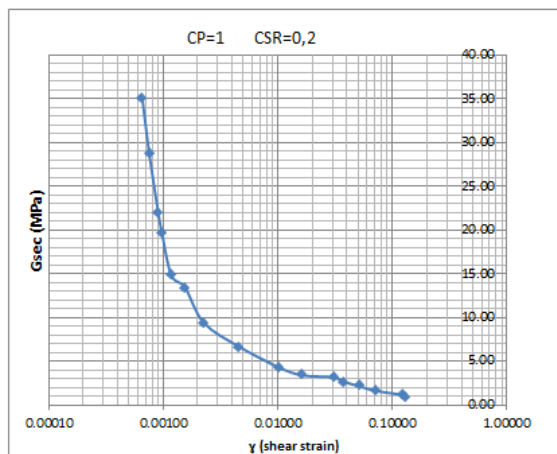


Figure A.18. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

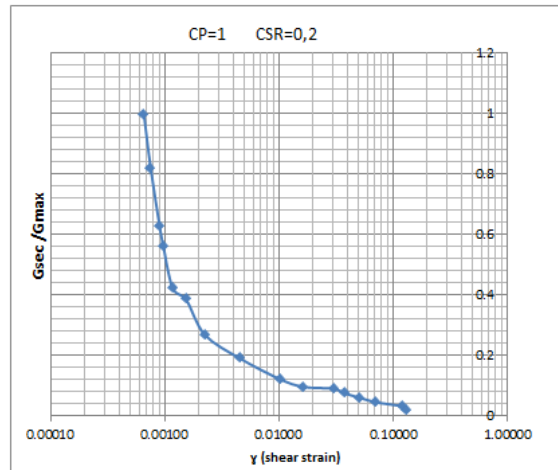


Figure A.19. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

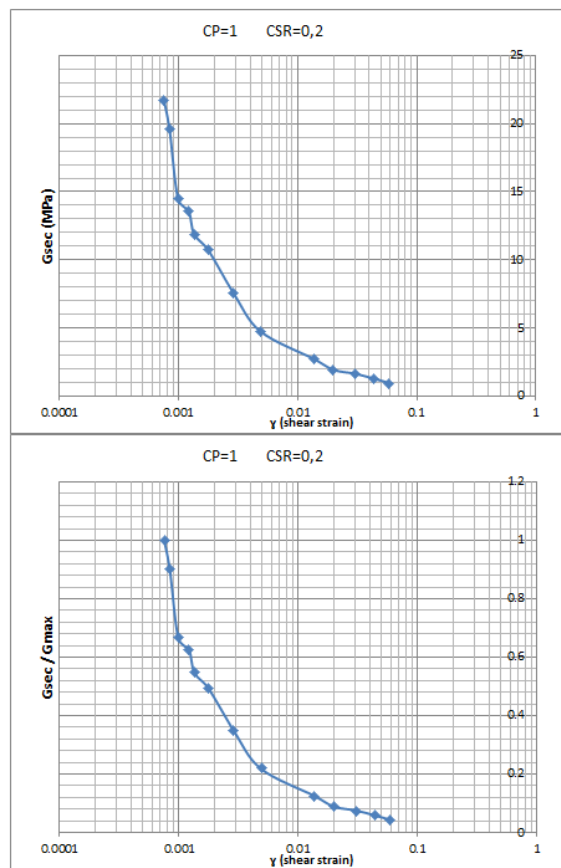


Figure A.20. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.

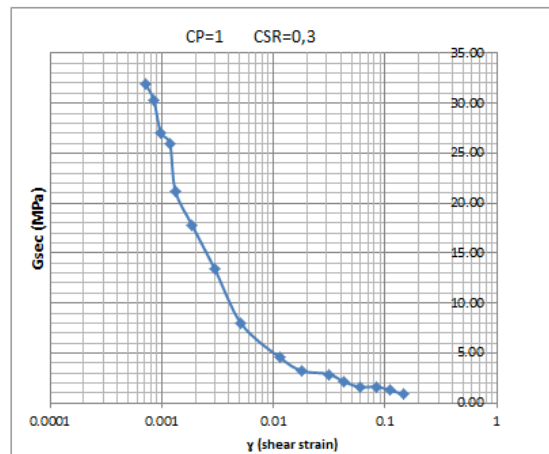


Figure A.21. Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

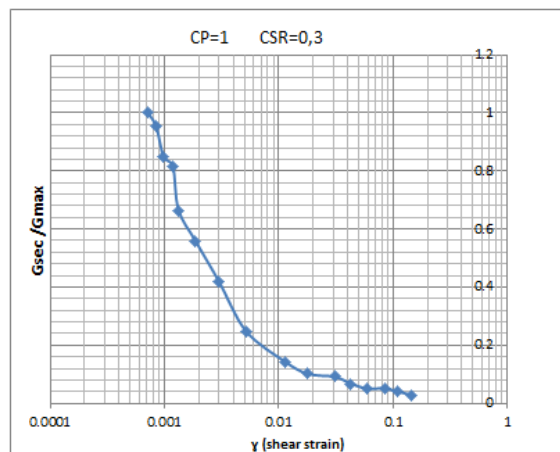


Figure A.22. Normalized Shear Modulus Reduction Curve for Şile Sand at Given Loading Conditions.

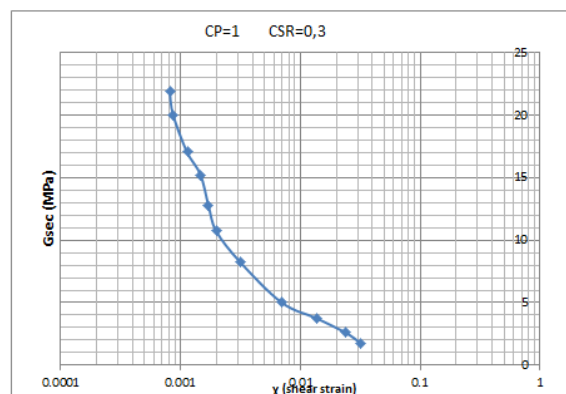


Figure A.23. Shear Modulus Reduction Curve for Well Graded Fly Ash Pellets at Given Loading Conditions.

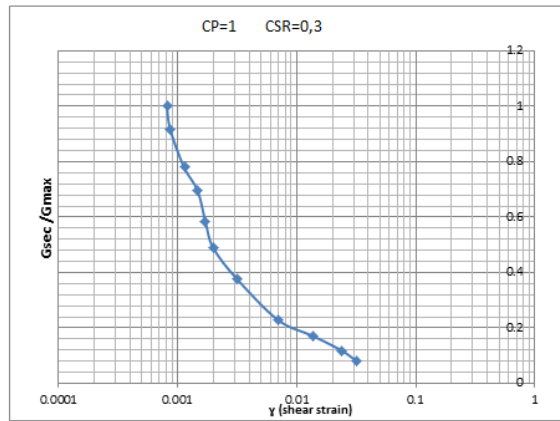


Figure A.24. Normalized Shear Modulus Reduction Curve for well Graded Fly Ash Pellets at Given Loading Conditions.

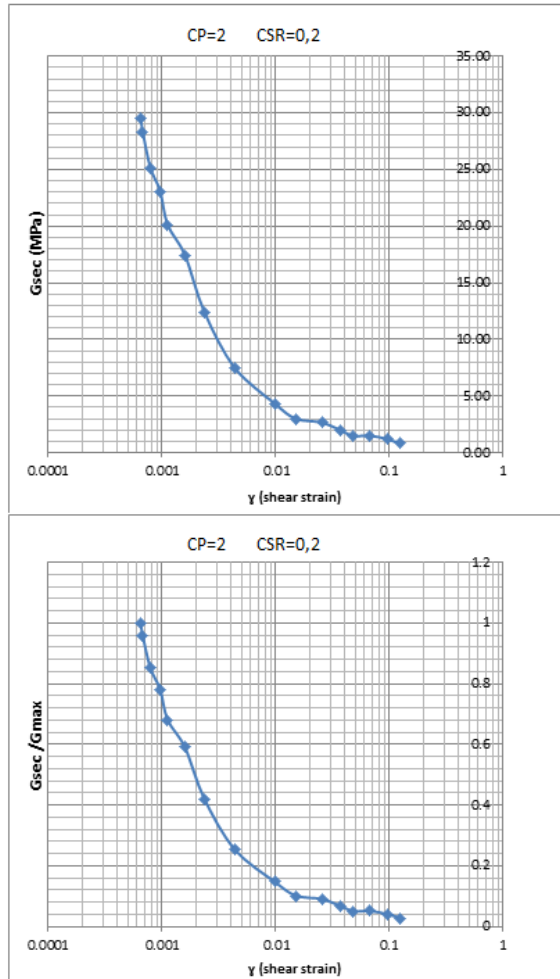


Figure A.25. Shear Modulus Reduction Curves for Şile Sand at Given Loading Conditions.

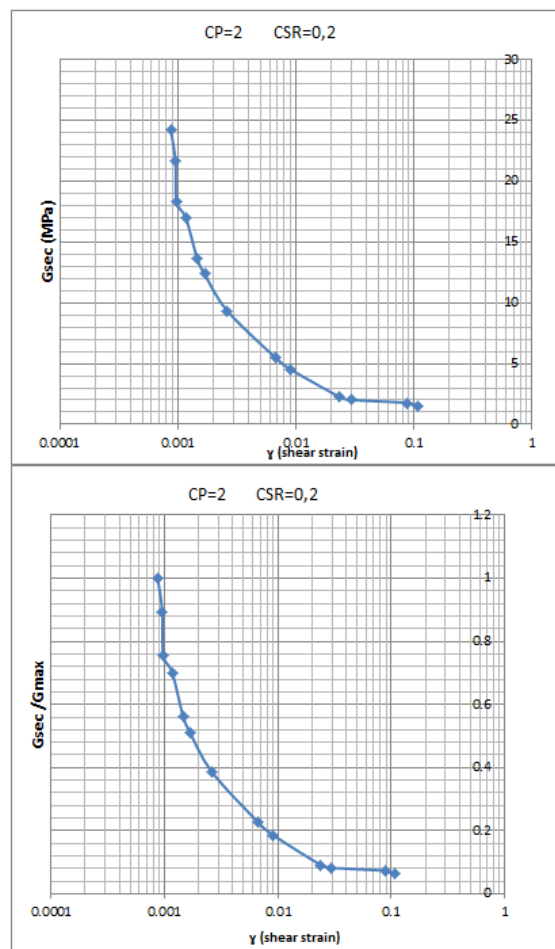


Figure A.26. Shear Modulus Reduction Curves for Well Graded Fly Ash Pellets at Given Loading Conditions.

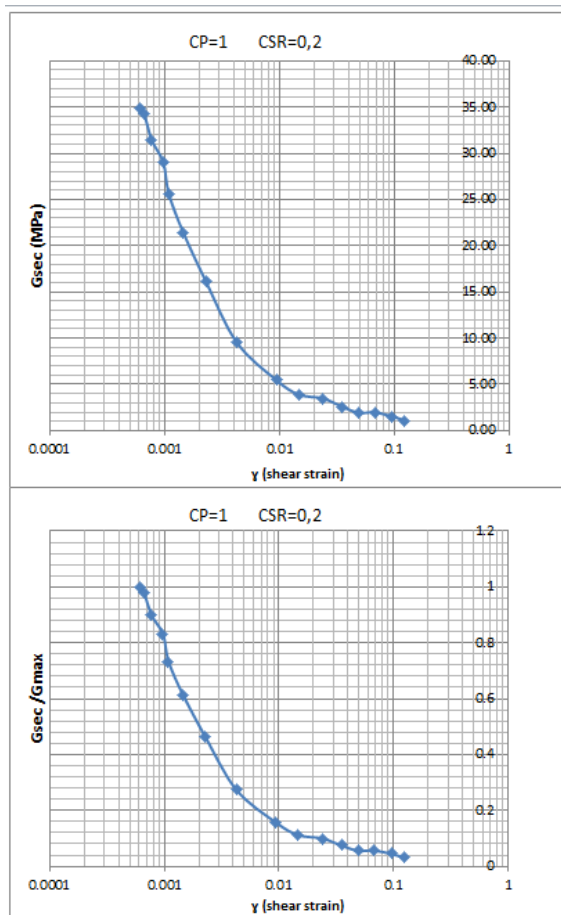


Figure A.27. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

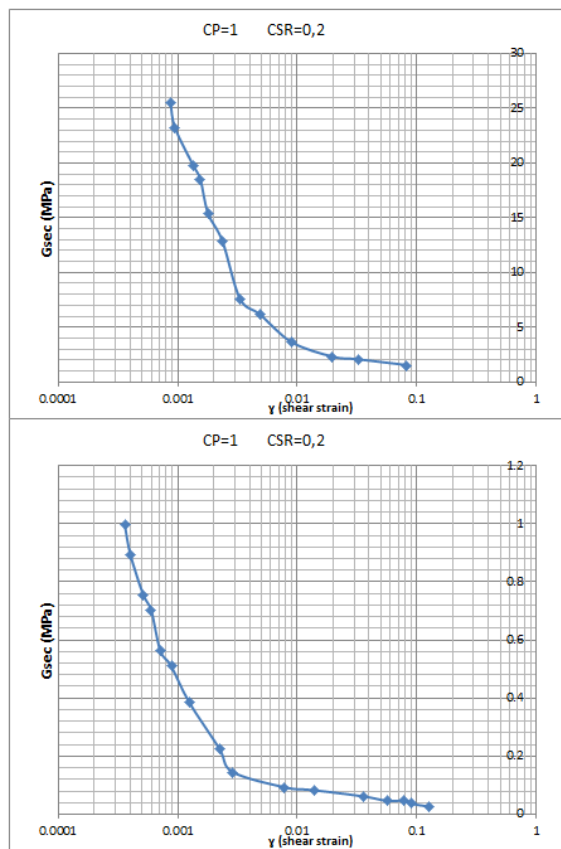


Figure A.28. Shear Modulus Reduction Curves for Poorly Graded Pellets at Given Loading Conditions.

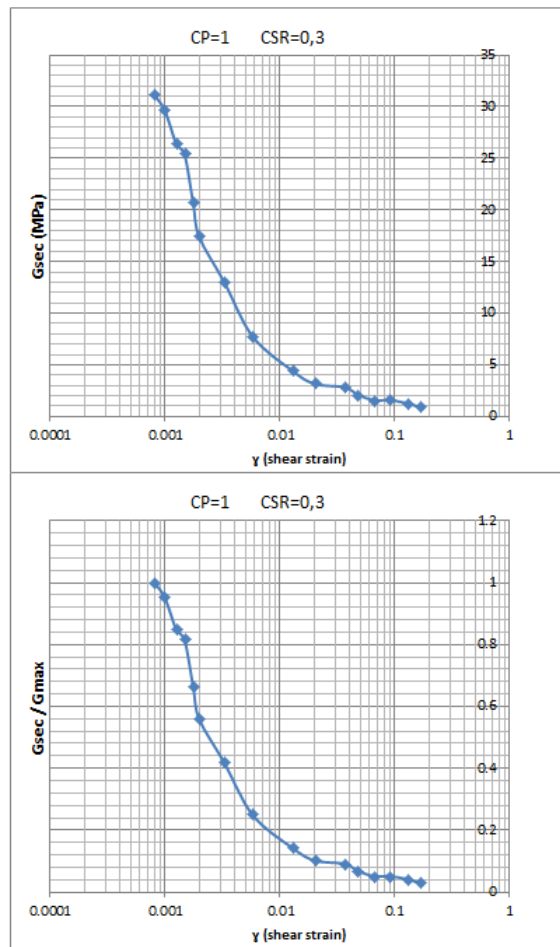


Figure A.29. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

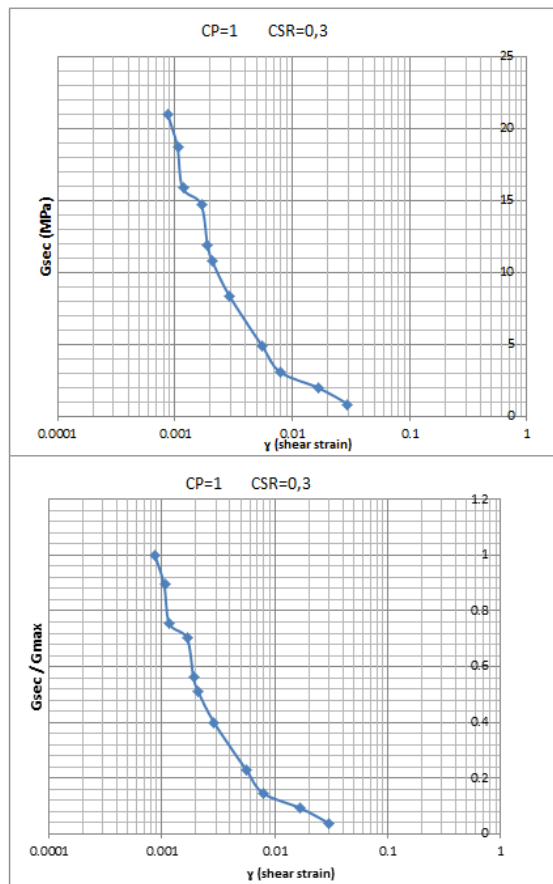


Figure A.30. Shear Modulus Reduction Curves for Poorly Graded Pellets at Given Loading Conditions.

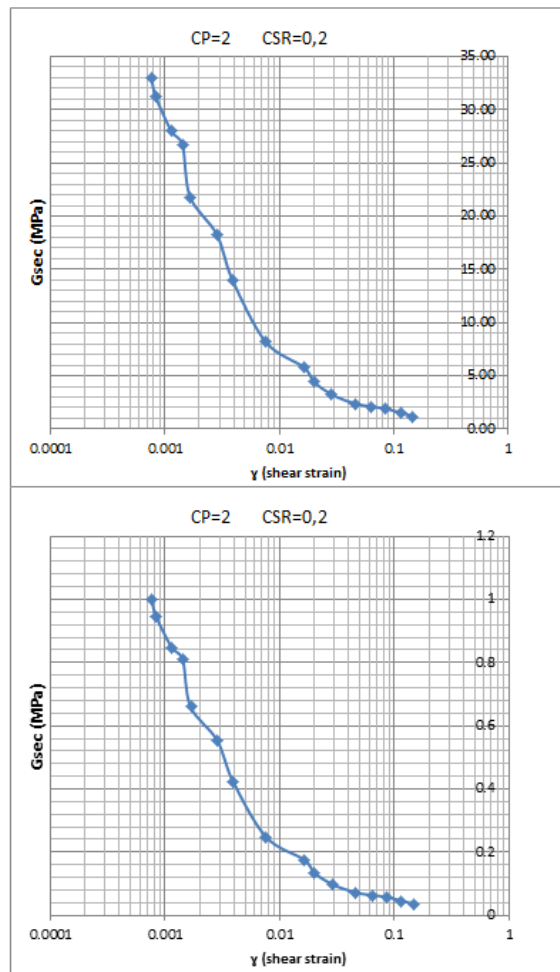


Figure A.31. Shear Modulus Reduction Curves for Kilyos Sand at Given Loading Conditions.

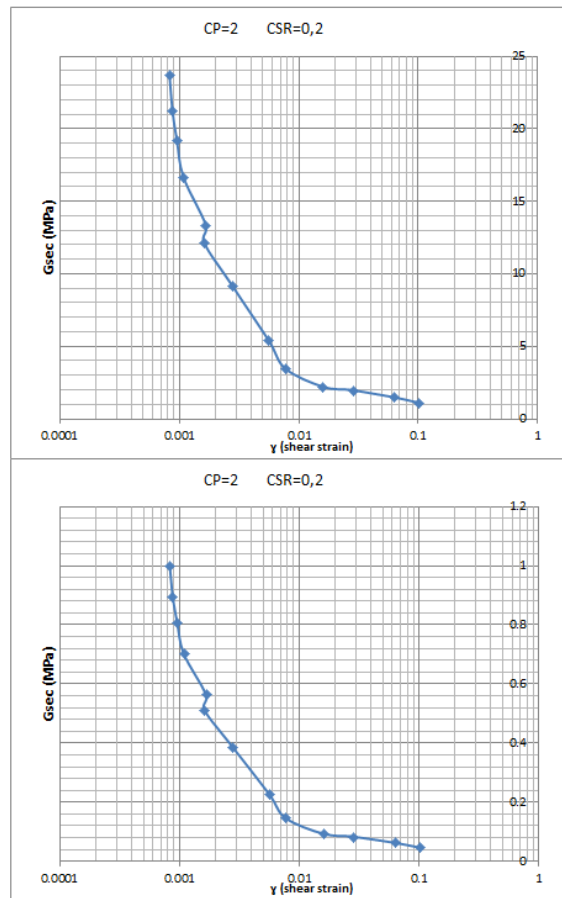


Figure A.32. Shear Modulus Reduction Curves for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

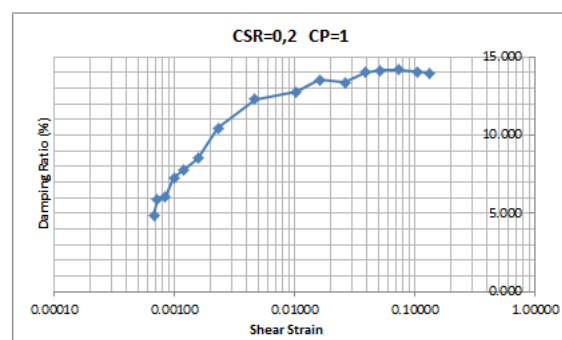


Figure A.33. Damping Ratio Values for Şile Sand at Given Loading Conditions.

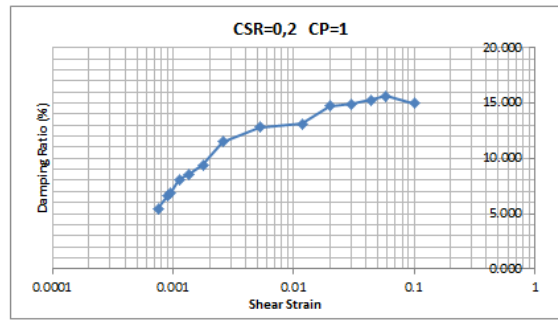


Figure A.34. Damping Ratio Values for Well Graded Fly Ash Pellets at Given Loading Conditions.

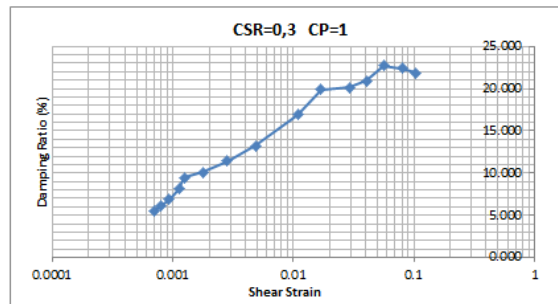


Figure A.35. Damping Ratio Values for Şile Sand at Given Loading Conditions.

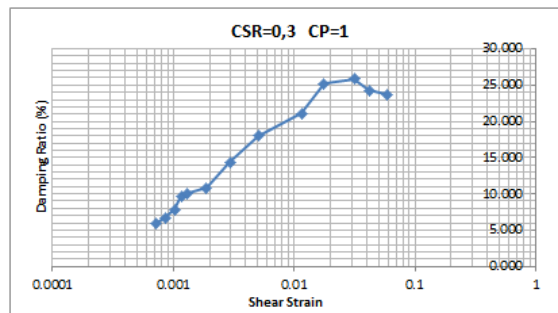


Figure A.36. Damping Ratio Values Forwell Graded Fly Ash Pellets at Given Loading Conditions.

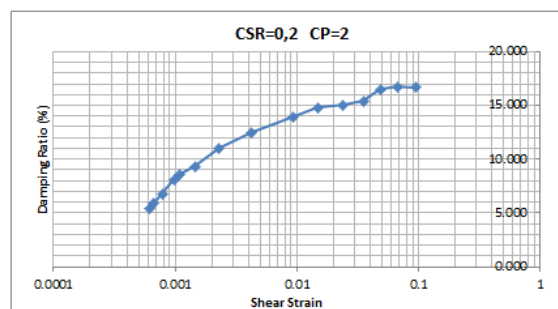


Figure A.37. Damping Ratio Values for Şile Sand At Given Loading Conditions.

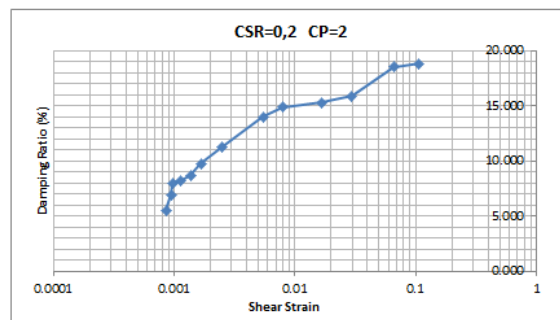


Figure A.38. Damping Ratio Values For Well Graded Fly Ash Pellets at Given Loading Conditions.

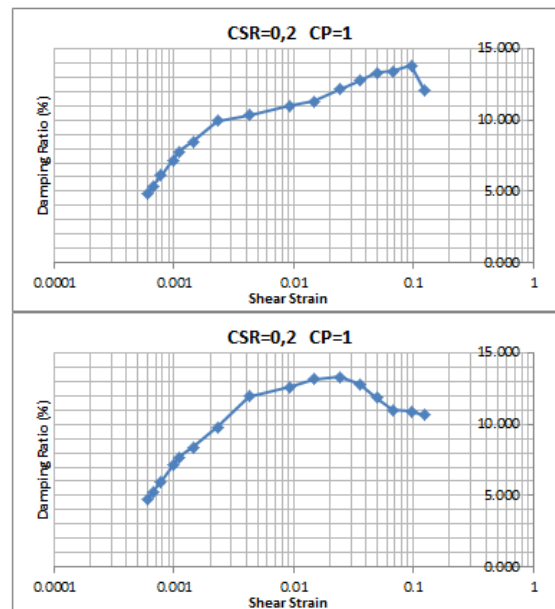


Figure A.39. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 1.

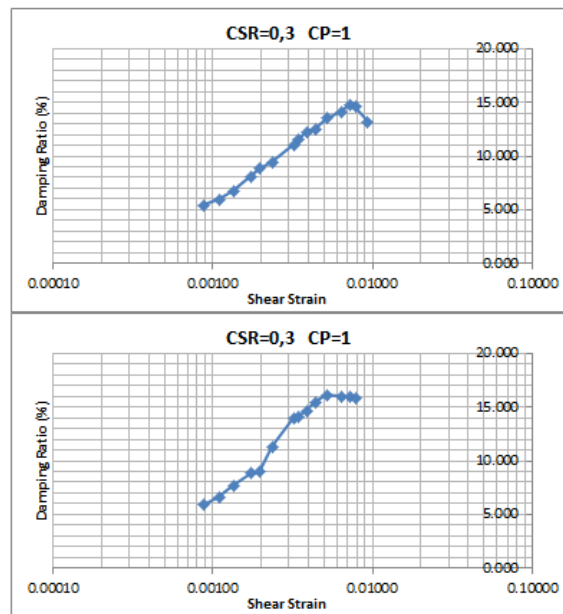


Figure A.40. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 2.

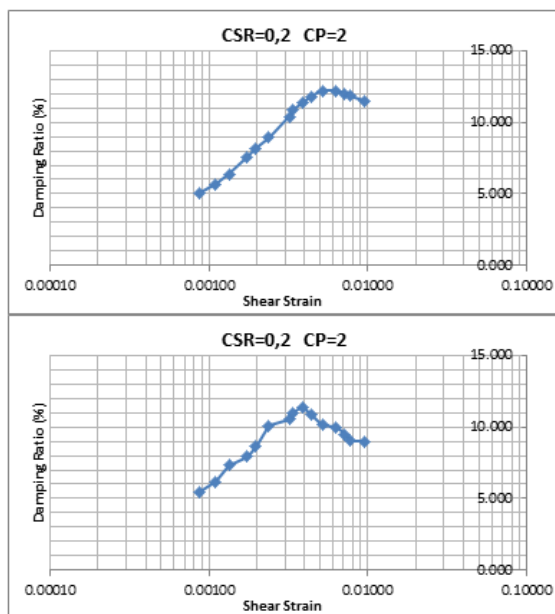


Figure A.41. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions 3.

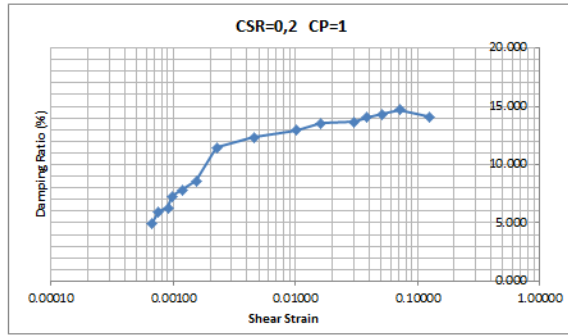


Figure A.42. Damping Ratio Values for Şile Sand at Given Loading Conditions.

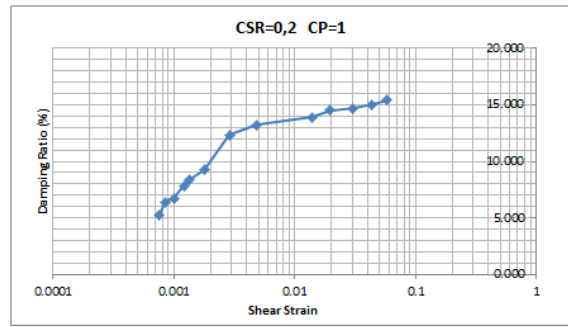


Figure A.43. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.

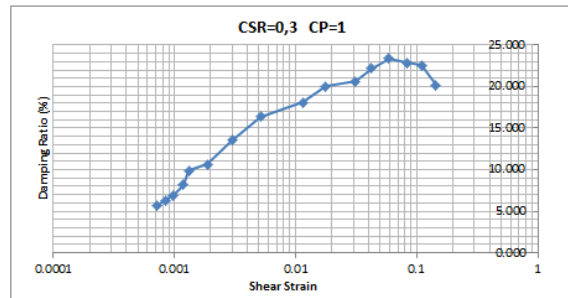


Figure A.44. Damping Ratio Values for Şile Sand at Given Loading Conditions.

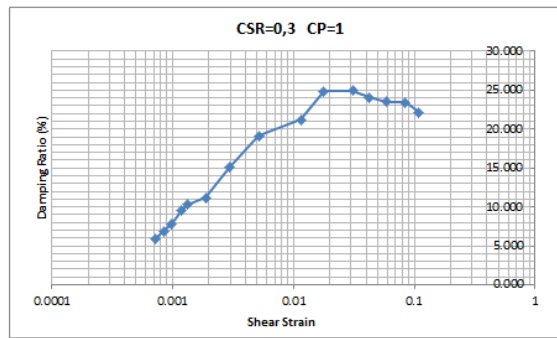


Figure A.45. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.

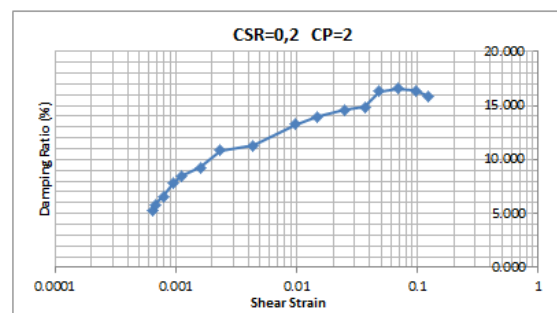


Figure A.46. Damping Ratio Values for Şile Sand at Given Loading Conditions.

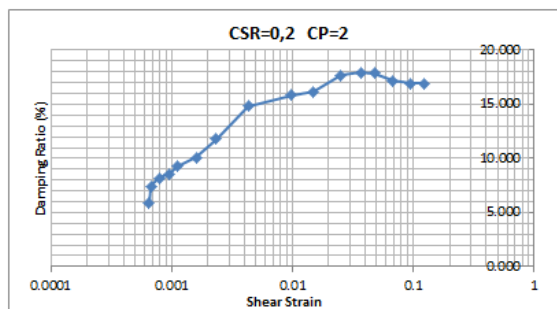


Figure A.47. Damping Ratio Values for well Graded Fly Ash Pellets at Given Loading Conditions.

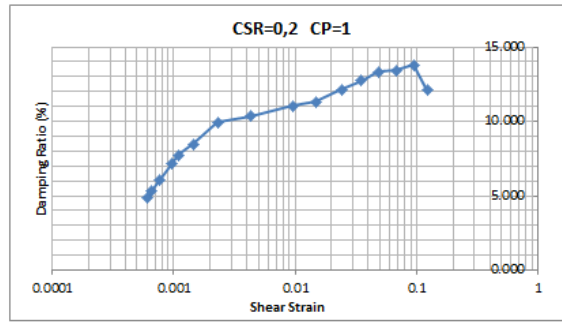


Figure A.48. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.

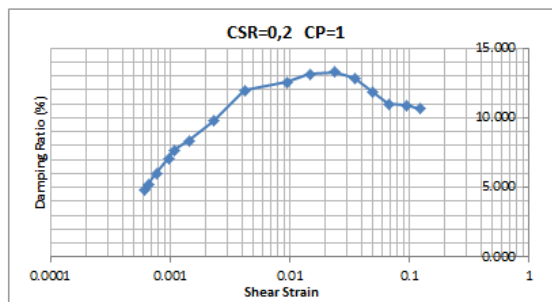


Figure A.49. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

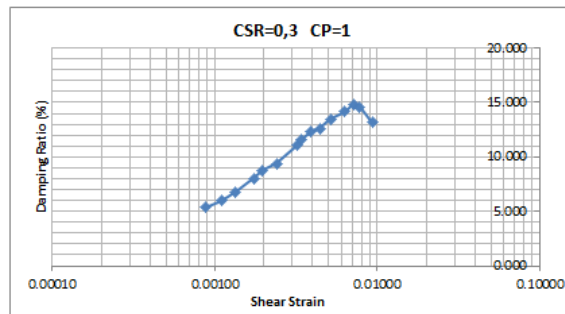


Figure A.50. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.

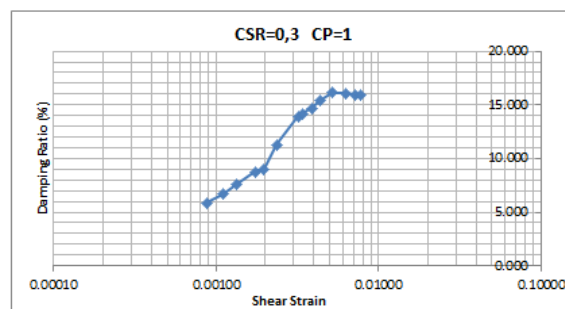


Figure A.51. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

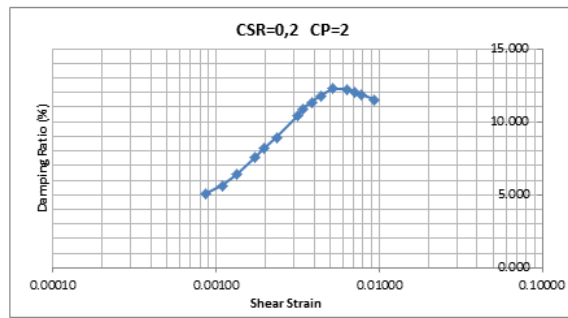


Figure A.52. Damping Ratio Values for Kilyos Sand at Given Loading Conditions.

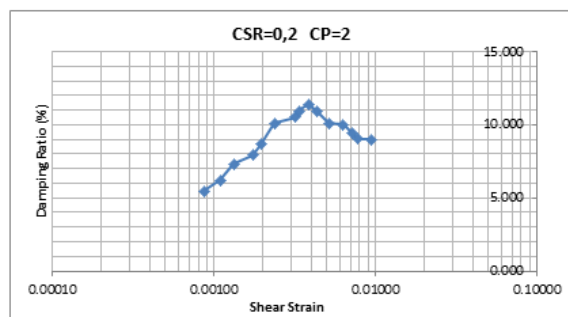


Figure A.53. Damping Ratio Values for Poorly Graded Fly Ash Pellets at Given Loading Conditions.

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