

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

A COMPARATIVE NUMERICAL ANALYSIS FOR LIQUEFACTION

Ph.D. THESIS

Mert TOLON

Department of Civil Engineering

Earthquake Engineering Programme

AUGUST 2013

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KARŞILAŞTIRMALI SAYISAL SIVILAŞMA ANALİZİ

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To my mother, father and all my supporters,

FOREWORD

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TABLE OF CONTENTS

	<u>Page</u>
FOREWORD	ix
TABLE OF CONTENTS.....	xi
ABBREVIATIONS	xv
LIST OF TABLES	xvii
LIST OF FIGURES	xix
LIST OF SYMBOLS	xxiii
SUMMARY	xxv
ÖZET.....	xxvii
1. INTRODUCTION.....	1
1.1 Background	1
1.2 Purpose of the Thesis	2
1.3 Methodology	2
1.4 Thesis Organization.....	3
2. LITERATURE REVIEW.....	5
2.1 Introduction	5
2.2 Literature Review on Definition of Liquefaction.....	5
2.2.1 Definition of liquefaction.....	5
2.2.2 Potentially liquefiable soils	10
2.2.3 Liquefaction susceptibility	12
2.2.3.1 Grain size distribution and soil types	14
2.2.3.2 Relative density	16
2.2.3.3 Earthquake loading characteristics.....	16
2.2.3.4 Vertical effective stress and over consolidation	16
2.2.3.5 Age and origin of the soils	16
2.2.3.6 Seismic strain history	17
2.2.3.7 Degree of saturation	17
2.2.3.8 Thickness of sand layer.....	17
2.2.4 Effects of liquefaction on soils.....	18
2.2.4.1 Liquefaction – Induced instability relationship.....	18
2.2.4.2 Liquefaction – Induced settlement relationship	20
2.2.4.3 Liquefaction – Earthquake relationship	21
2.2.4.4 Flow liquefaction surface and excess pore pressure relationship	21
2.2.5 Evaluation of initiation of liquefaction	23
2.2.5.1 Cyclic stress approach.....	23
2.2.5.2 Cyclic strain approach.....	28
2.2.5.3 Probabilistic approach.....	29
2.2.6 In-Situ test methods	29
2.2.6.1 Standard penetration test.....	30
2.2.6.2 Cone penetration test.....	35
2.2.6.3 Shear wave velocity tests	39

2.2.7 The return period of soil liquefaction.....	44
2.2.8 Mitigation procedures of liquefaction	45
2.3 Literature Review on Liquefaction Analysis Methods.....	51
2.3.1 Overview	51
2.3.2 Artificial neural network analysis	51
2.3.3 Probabilistic neural network analysis.....	51
2.3.4 Logistic regression analysis	53
2.3.5 Fuzzy neural network analysis	54
2.3.6 Adaptive resonance theory based network analysis.....	54
2.3.7 Instance based learning model analysis	56
2.3.8 Evolutionary polynomial regression	57
2.3.9 Genetic expression programming.....	59
2.3.10 Finite element analysis	60
2.4 Neural Network Algorithm.....	61
2.5 Finite Difference Method	65
2.6 Computer Programs for Liquefaction Analysis.....	66
2.6.1 Computational tools for liquefaction analysis.....	66
2.6.2 Finite difference based liquefaction analysis	72
2.6.3 Neural network based liquefaction analysis.....	75
2.7 Discussion.....	82
3. CASE STUDY.....	83
3.1 General	83
3.2 Turkey, Istanbul Microzonation Project.....	83
3.2.1 Data description.....	84
3.2.2 Site characterization	86
3.2.2.1 General soil properties of Kurbağlıdere region.....	89
3.2.2.2 Dynamic soil properties	89
3.2.3 Earthquake information for Istanbul	91
4. DEVELOPMENT OF MODELS.....	93
4.1 Finite Difference Model	93
4.1.1 General	93
4.1.2 Mathematical formulation	93
4.1.3 Soil generation.....	94
4.1.4 Grid generation.....	98
4.1.5 Boundary conditions	101
4.1.6 Initial conditions.....	102
4.1.7 Constitutive model	103
4.1.8 Earthquake motions.....	103
4.1.8.1 Filtering	105
4.1.8.2 Baseline correction.....	107
4.1.9 Dynamic pore pressure generation.....	107
4.2 Neural Network Model.....	110
4.2.1 General	110
4.2.2 General modeling parameters.....	110
4.2.2.1 Modeling of soil elements	110
4.2.2.2 Modeling of the earthquake motion	111
4.2.2.3 Modeling of the water	111
4.2.2.4 Datasets used in neural network models	111
4.2.3 Architecture design	112
4.2.4 Training phase	113

4.2.5 Learning phase.....	114
4.2.6 Validation phase.....	114
5. RESULTS	117
5.1 Introduction	117
5.2 Kurbağlıdere Region FDM based Model Results	117
5.3 FDM Model Results of 1D – 2D and 3D geometries for Zone 1 and Zone 8	121
5.4 Kurbağlıdere Region NN based Model Results	123
5.4.1 BPNN result.....	123
5.4.2 GRNN result.....	128
6. CONCLUSION AND RECOMMENDATIONS	135
6.1 Summary	135
6.2 Conclusion.....	135
6.3 Recommendations for Future Works	135
REFERENCES	139
APPENDICES	147
APPENDIX A	149
APPENDIX B	153
APPENDIX C	165
APPENDIX D	175
APPENDIX E.....	181
APPENDIX F.....	183
APPENDIX G	195
CURRICULUM VITAE.....	207

ABBREVIATIONS

AI	: Artificial Intelligence
ANN	: Artificial Neural Network
App	: Appendix
ART	: Adaptive Resonance Theory
BPNN	: Back Propagation Neural Network
BPT	: Becker Penetration Test
CHST	: Cross-Hole Seismic Test
CPT	: Cone Penetration Test
CRR	: Cyclic Resistance Ratio
CSR	: Cyclic Stress Ratio
CVR	: Critical Void Ratio
DHST	: Down Hole Seismic Test
FDM	: Finite Difference Method
FEM	: Finite Element Method
FNN	: Fuzzy Neural Networks
FS	: Factor of Safety
FSL	: Flow Liquefaction Surface
GA	: Genetic Algorithm
GEP	: Genetic Expression Programming
GMDH	: Group Method of Data Handling
GRNN	: General Regression Neural Network
IBL	: Instance Based Learning
LL	: Liquid Limit
MSF	: Magnitude Scaling Factors
NN	: Neural Network
PCPT	: Piezo Cone Penetration Test
PDE	: Partial Differential Equations
PI	: Plasticity Index
PNN	: Probabilistic Neural Network
PSHA	: Probabilistic Seismic Hazard Analysis
SASW	: Spectral Analysis of Surface Waves
SCPT	: Seismic Cone Penetration Test
SPT	: Standard Penetration Test
SSL	: Steady State Line
Vs	: Shear Wave Velocity

LIST OF TABLES

	<u>Page</u>
Table 2.1: Formulated Chinese criteria proposed by Seed and Idriss.	10
Table 2.2: Modified Chinese criteria proposed by Andrews and Martin.	11
Table 2.3: Comparison of SPT based evaluation methods for liquefaction initiation - part I	27
Table 2.4: Comparison of SPT based evaluation methods for liquefaction initiation - part II	28
Table 2.5: Comparison of various field test for assessment of liq. pot.	29
Table 2.6: Recommended corrections for the SPT equipment energy procedures ..	35
Table 2.7: Comparison of in-situ tests	44
Table 2.8: Liquefaction counter measures.	46
Table 2.9: General methods for mitigations of seismic soil liq. hazard.	50
Table 2.10: Regression constants of bayesian mapping function.	53
Table 2.11: ART based neural network architectures proposed.	55
Table 3.1: Content of geological studies.	85
Table 3.2: Laboratory test and experiments.	85
Table 3.3: Total samples taken for case	85
Table 3.4: Coordinates of study area	88
Table 3.5: The earthquakes occurred in the past at the region	92
Table 4.1: The earthquakes occurred in past at the region.	96
Table 4.2: Mesh sizes created for all zones I	101
Table 4.3: Calculated finn constants for zone 1	109
Table 4.4: Distribution of the data for liquefaction occurrence	110
Table 4.5: Distribution of the data among phases.	110
Table 4.6: Used soil parameters in NN model	111
Table 4.7: Earthquake motion parameters in NN model	111
Table 4.8: Water parameters in NN model.	111
Table 4.9: Summary of SPT liquefaction case records in NN model.	112
Table 4.10: Variable types and min, max and mean values of parameters	113
Table 4.11: BPNN based model chosen layer parameter values.	114
Table 4.12: GRNN based model chosen layer parameter values	115
Table 5.1: Success rates of models for all zones.	119
Table 5.2: Forecasted Ru values by using BPNN.	123
Table 5.3: Success rate of BPNN model	124
Table 5.4: The contribution factors for BPNN model	125
Table 5.5: Forecasted Ru values by using GRNN	128
Table 5.6: Success rate of GRNN model	128
Table B.1: Calculated and defined soil properties for FDM models	153
Table C.1: Finn constants calculated for all zones in FDM models	165

Table E.1: Evaluated pore pressure ratio's for all zones.....	181
Table G.1: Input parameters for neural network models.	195

LIST OF FIGURES

	<u>Page</u>
Figure 2.1 : Liquefaction phenomena.	6
Figure 2.2 : Soil loses its contact force with increasing of pore water pressure and transforms to the liquefaction state.	7
Figure 2.3 : Examples of liquefaction damage	8
Figure 2.4 : Liquefaction occurrence with CRR (CSR) to SPT-N (qc) graph.....	10
Figure 2.5 : Criteria for liquefaction susceptibility of fine-grained sediments proposed by Seed	11
Figure 2.6 : Criteria for differentiating between sand-like and clay-like sediment behavior proposed by Boulanger and Idriss	12
Figure 2.7 : CVR line as a boundary between loose and dense states.	14
Figure 2.8 : Zone of initial stress conditions susceptible to flow liquefaction.	22
Figure 2.9 : The susceptible area of cyclic mobility.	23
Figure 2.10 : Potential zone of liquefaction.	27
Figure 2.11 : Photograph of setup for SPT with energy measurements at site.	31
Figure 2.12 : Typical energy measurement plot for SPT boring.	31
Figure 2.13 : Typical setup of SPT.	32
Figure 2.14 : Relationship between normalized SPT count and CRR.	34
Figure 2.15 : Recommended C_R values.	34
Figure 2.16 : Standard cone used for cone penetration test	36
Figure 2.17 : Relationship between normalized CPT tip resistance and CRR	37
Figure 2.18 : Fines correction by CPT friction ratio to CPT resistance graph.	39
Figure 2.19 : Field setup for SASW tests.....	42
Figure 2.20 : Shear wave velocity based liquefaction triggering correlation.	43
Figure 2.21 : Typical soil improvement techniques for densifying loose saturated sands to prevent earthquake induced liquefaction.	48
Figure 2.22 : Normalized coefficient of volume compressibility versus pore pressure ratio for sands at various relative densities	49
Figure 2.23 : SPT based probability curves using bayesian map.func.	53
Figure 2.24 : Fuzzy ARTMAP architecture.....	56
Figure 2.25 : Schematic diagram of a neuron's network.	62
Figure 2.26 : Neural networks structure.....	62
Figure 2.27 : Basic explicit calculation cycle in FLAC3D.	74
Figure 2.28 : Finite difference mesh in Flac 3D	75
Figure 2.29 : Advanced options screen in Neuroshell2.	78
Figure 2.30 : Rules Module in Neuroshell2.....	78
Figure 2.31 : Back propagation architecture design module in Neuroshell2.....	79
Figure 2.32 : Network architecture.	79
Figure 3.1 : SPT log information in Kurbağlıdere region.....	86

Figure 3.2 :	An example SPT log file in Kurbağlıdere region at Zone 1.....	87
Figure 3.3 :	Istanbul microzonation project areas, by Istanbul metropolitan municipality.....	87
Figure 3.4 :	Chosen case study area of Kurbağlıdere, Kadıköy, Istanbul.....	88
Figure 3.5 :	Example map of liquefaction potential for Istanbul done by IBB.....	90
Figure 3.6 :	Liquefaction potential map of case study area at Kurbağlıdere.	90
Figure 3.7 :	Seismic risk map of Istanbul.	92
Figure 4.1 :	An example maximum unbalanced force that reaches zero at static analysis.....	96
Figure 4.2 :	Soil property groups defined for all 250m – 250 m zone in region	97
Figure 4.3 :	Soil property groups for Zone 1 in FDM model.....	97
Figure 4.4 :	Density values defined for soil layers in Zone 1	98
Figure 4.5 :	An 8-node zone with 2 overlays of 5 tetrahedra in each overlay.	99
Figure 4.6 :	Zonations of the Kurbağlıdere area in FDM model	99
Figure 4.7 :	Meshes created for all zones in FDM model.....	100
Figure 4.8 :	Meshes created for Zone 1 in FDM model.....	100
Figure 4.9 :	Boundary conditions defined for all zones in FDM models.....	102
Figure 4.10 :	Seismic input to FLAC3D	104
Figure 4.11 :	Acceleration characteristics given as input motion to FLAC3D	104
Figure 4.12 :	Motion given in the layer of rock at zone 1 in FDM model.....	105
Figure 4.13 :	Summary of FDM based liquefaction analysis done with FLAC3D.	109
Figure 4.14 :	Different architecture types tested for NN models.....	113
Figure 4.15 :	An example hidden layer properties for BPNN architecture type....	114
Figure 4.16 :	An example hidden layer properties for GRNN architecture type.. ..	115
Figure 4.17 :	Summary of NN based liquefaction analysis done with Neuroshell.	115
Figure 5.1 :	The pore pressure values at layers in zone 1 for static analysis.....	117
Figure 5.2 :	The pore pressure values at layers in zone 1 for dynamic analysis....	118
Figure 5.3 :	Chosen points at zones for FDM 3D models.....	118
Figure 5.4 :	Pore Pressure Ratio - Step Graph for Zone 1, Point 3, Layer 3 as a Silty Sand layer.	119
Figure 5.5 :	Pore Pressure Ratio / Depth Graph for Zone 1	120
Figure 5.6 :	Pore pressure ratio-depth graph in zone 1 for 1D,2D,3D FDM.	121
Figure 5.7 :	Pore pressure ratio-depth graph in zone 8 for 1D,2D,3D FDM.	122
Figure 5.8 :	Error – epochs elapsed graph of BPNN model.....	125
Figure 5.9 :	The contribution factors graph for BPNN model	125
Figure 5.10 :	BPNN and FDM model results comparisons for Zone 10.....	126
Figure 5.11 :	BPNN and FDM model results comparisons for Zone 11.....	127
Figure 5.12 :	Error – epochs elapsed graph of GRNN model	130
Figure 5.13 :	GRNN and FDM model results comparisons for Zone 10.....	131
Figure 5.14 :	GRNN and FDM model results comparisons for Zone 11.....	132
Figure 5.15 :	GRNN, BPNN and FDM 3D model results comparisons zone 10....	133
Figure 5.16 :	GRNN, BPNN and FDM 3D model results comparisons zone 11....	134
Figure A.1 :	Sample SPT soil profile used for zone 1.	149
Figure D.1 :	Soil property groups for zone 1.	175
Figure D.2 :	Soil property groups for zone 2.	175
Figure D.3 :	Soil property groups for zone 3.	176
Figure D.4 :	Soil property groups for zone 4.	176
Figure D.5 :	Soil property groups for zone 5.	177

Figure D.6 : Soil property groups for zone 6.	177
Figure D.7 : Soil property groups for zone 7.	178
Figure D.8 : Soil property groups for zone 8.	178
Figure D.9 : Soil property groups for zone 9.	179
Figure D.10 : Soil property groups for zone 10.	179
Figure D.11 : Soil property groups for zone 11.	180
Figure F.1 : Evaluated pore pressure ratio – depth graph of zone 1.	183
Figure F.2 : Evaluated pore pressure ratio – depth graph of zone 2.	184
Figure F.3 : Evaluated pore pressure ratio – depth graph of zone 3.	185
Figure F.4 : Evaluated pore pressure ratio – depth graph of zone 4.	186
Figure F.5 : Evaluated pore pressure ratio – depth graph of zone 5.	187
Figure F.6 : Evaluated pore pressure ratio – depth graph of zone 6.	188
Figure F.7 : Evaluated pore pressure ratio – depth graph of zone 7.	189
Figure F.8 : Evaluated pore pressure ratio – depth graph of zone 8.	190
Figure F.9 : Evaluated pore pressure ratio – depth graph of zone 9.	191
Figure F.10 : Evaluated pore pressure ratio – depth graph of zone 10.	192
Figure F.11 : Evaluated pore pressure ratio – depth graph of zone 11.	193

LIST OF SYMBOLS

C_E	: Hammer Efficiency Correction
C_B	: Borehole Diameter Correction
C_S	: Sampler Correction
c	: Cohesion
D_r	: Relative Density
E_m	: Actual Hammer Energy
E_f	: Theoretical Free-Fall Hammer Energy
f	: Fundamental Frequency of Natural Mode of System Oscillation
G	: Elastic Shear Modulus
$G(\gamma_{cyc})$	: Shear Modulus of the Soil at $\gamma = \gamma_{cyc}$
K	: Elastic Bulk Modulus
$N_{1,60}$	: Normalized Penetration Resistance Number
PL	: Probability of Liquefaction
S_r	: Residual Strength
r_u	: Pore Pressure Ratio
u	: Pore Water Pressure
q_c	: Tip Resistance
w_c	: Water Content
σ'	: Effective Stress
σ	: Total Stress
ρ	: Mass Density
ψ	: Dilation Angle
Φ	: Internal Friction Angle
σ_t	: Tension Limit
λ	: Longest Wavelength of Natural Mode of Oscillation of a System
ϵ_{vd}	: Volumetric Strain
$\Delta\epsilon_{vd}$	: Incremental Volumetric Strain
Δu	: Increase in Residual Excess Pore Water Pressure for the Cycle

A COMPARATIVE NUMERICAL ANALYSIS FOR LIQUEFACTION

SUMMARY

Soil liquefaction is the phenomenon of temporary loss of shear in saturated cohesionless soils under the influence of vibrations caused by earthquakes. Due to the complexity of liquefaction problems, such as nonlinearity of responses, sudden phase transition from solid to liquid behavior, material instability, interaction and relative movement between the porous soil skeleton and interstitial water, limitations on experimental and numerical techniques, material model formulation and so on reliable and accurate predictive methods need to be developed.

The extensive loss of lives and civil infrastructure system (buildings, bridges, highways and airports etc.) caused by liquefaction emphasize the need for strong and reliable methods for evaluating the liquefaction potential of sites. Conventional methods for evaluating liquefaction potential are based on correlating data from soil Standard Penetration Test (SPT) with a parameter that represents the seismic loading on the soil during an earthquake.

This study aims to explore the best model for the prediction of liquefaction potential by comparing the finite difference model and neural network model for the chosen case study area which is Kurbağlıdere region at Kadıköy, Istanbul, Turkey.

The methodology of the research are to develop finite difference model in 1D, 2D and 3D geometries to predict liquefaction potential of a site and also to develop a neural network model to predict the liquefaction potential of a site. In the developed models, the aim is to compare simplified methods results with finite difference method based model results and neural network algorithm based model results.

In the developed finite difference method (FDM) models for evaluating liquefaction occurrence of soil, it is seen that the developed FDM model is a useful model for forecasting the liquefaction potentials of a point in the case study area which we do not have any information. The comparison of simplified equations method results and the FDM model results are close and meaningful and the success rates of the developed models are meaningful too with an average success rate of 92%.

In the developed back propagation neural network and general regression neural network models use of the SPT data for training and testing is done. Various models are examined and the model with the highest success rate (92%) is presented. In these neural network models, the most effective parameter for liquefaction occurrence is determined and it is seen that friction angle of soils and SPT-N corrected blowcount values are the most effective parameters for liquefaction potentials of soils.

As a summary, the results obtained by the two proposed models demonstrate the liquefaction potential of regions and encourage the use of these applications for forecasting studies.

KARŞILAŞTIRMALI SAYISAL SIVILAŞMA ANALİZİ

ÖZET

Suya doygun, gevşek kum veya kumlu zeminler, tekrarlı yükler etkisinde, sıkışma ve hacim daralması eğilimi gösterirler. Bu eğilim, drenajın olmadığı koşullarda, boşluk suyu basıncını artırır. Tekrarlı yükler kum tabakası içindeki boşluk suyu basıncının artmasını desteklediği zaman, toplam gerilme, boşluk suyu basıncına eşit değere ulaşabilir.

Bu durumda, kohezyonsuz zemin kayma direncini kaybeder ve bir sıvı gibi davranarak büyük yer değiştirmelerine maruz kalır. Böylece sıvılaşma evresine geçilmiş olur.

Zemin sıvılaşmasının etkisinin görüldüğü örnekler; 1964 Niigata, 1920 California Calvers, 1938 Montana Fort Peck, 1948 Fukui, 1971 California San Fernando, 1964 Alaska Anchorage, 1980 Mino-Owari olarak sıralanabilir. Ülkemizdeki son büyük örnek de 1999 Marmara depremi sıvılaşmalarıdır.

Zemin sıvılaşması özetle, deprem sebebiyle oluşan titreşimlerin doymuş kohezyonsuz zeminlerde taşıma gücünün geçici olarak kaybedilmesi şeklinde tanımlanmaktadır. Sıvılaşma sorunlarının karmaşıklığı nedeniyle, yanıtların doğrusal olmaması gibi, malzeme istikrarsızlıkları, deneysel ve sayısal teknik sınırlamaları, malzeme modeli formülasyonu zorlukları görüldüğü sebeple çok güvenilir ve doğru tahmin yöntemleri geliştirilmelidir.

Sıvılaşma olgusunun neden olduğu can kayıpları ve altyapı sistemlerinde meydana gelen hasarlar, bölgelere ait sıvılaşma potansiyellerinin önceden belirlenip değerlendirilmesinde güçlü ve güvenilir metotlara ihtiyaç duyulmasına neden olmuştur.

Zemin sıvılaşması, son depremlerde de (Adapazarı, Düzce, Türkiye; Chi-Chi, Taiwan, 1999) örneklediği üzere önemli hasara sebep olmuştur. Sıvılaşmanın mekanizması ve doğurduğu sonuçların anlaşılmasına yönelik çalışmalar, 1964 yılında meydana gelen Niigata - Japonya ve Büyük Alaska - A.B.D. depremleri sonrası daha da hızlanmıştır. Son kırk yılda bu alanda önemli ilerlemeler kaydedilmiştir. Bugün zemin sıvılaşması konusu geoteknik mühendisliğinde branşında olgunlaşmış ve de kendi özel uygulamaları olan bir alan olarak karşımıza çıkmaktadır.

Zeminde sıvılaşma oluşumunu etkileyen bir çok faktör vardır. Laboratuvar deney sonuçları yanında arazi gözlemleri ve çalışmalarına dayalı olarak, sıvılaşmayı etkileyen en önemli faktörler geçmişteki bazı çalışmalarda açıklanmıştır ve yeni çalışmalar ile açıklanmaya halen devam etmektedir. Deprem şiddeti ve süresi, ivme, yeraltı su seviyesi, zemin tipi, zeminin rölatif sıkılığı (D_r) ve benzeri etkin parametreler bu faktörler arasındadır.

Bu çalışmada ise sıvılaşma potansiyelinin tahmin edilmesinde en iyi modelin geliştirilmesi amacı ile sonlu farklar metoduna dayalı model ile yapay sinir ağları

metoduna dayalı modellerin sonuçları karşılaştırılmıştır. Kurbağlıdere, Kadıköy, İstanbul, Türkiye de gerçekleştirilmiş olan mikrobölgeleme çalışmasından faydalanılarak standart penetrasyon deneyi sondajlarından SPT-N değerleri hesaplanmış ve zemin parametreleri hesaplanarak bölgenin üç boyutlu sonlu farklar metoduna dayanan model ile gerçekleştirilmiştir. Geliştirilen sonlu farklar metoduna dayalı modellerde 1 boyutlu, iki boyutlu ve üç boyutlu geometrilerde modellemelerin sonuçları karşılaştırılmıştır. Model sonuçları anlamlı değerler vermiş olup, %92'lik bir modelleme başarısı görülmüştür.

Sonrasında ise, bölgede beklenen bir deprem etkisi altında zeminin boşluk basıncı, toplam gerilmesi ve efektif gerilmeleri dikkate alınarak, bu deprem altında zeminin üç boyutlu sıvılaşma riski potansiyelleri bulunmuştur. Bu sonuçların karşılaştırılması için, geliştirilen sonlu farklar modelinde kullanılan etkin parametreler (girdi) ve zemin sıvılaşması potansiyelini temsil eden boşluk basıncı oranı (çıkı) ile yapay sinir ağları metoduna bağlı model geliştirilmiştir.

Bu yapay sinir ağları modellerinde, eğitime ve test etme aşamalarında SPT 'lerden elde edilmiş data kullanılmıştır. Çeşitli modeller denenmiş ve en yüksek başarı oranının elde edildiği modeller olan geri yayılma (back propagation) ile genel regresyon (general regression) mimarilerine dayanan yapay sinir ağları modelleri geliştirilmiştir. Seçilen bölgede sıvılaşma analizlerinin model ile tahmin edilebilmesi için dikkate alınan zemin parametreleri girdi olarak tanımlanmış, boşluk basıncı oranı (r_u) çıkıtları model sonucu olarak tahmin edilmesi sağlanmıştır.

Geliştirilen yapay sinir ağları modellerinde sonuçların tutarlılıkları modellerin başarı oranları hesaplanarak tayin edilmiştir. Bunun için gerek modelin sonuçlarında programca hesaplanan başarı oranları alınmış gerek ise sonlu farklar metoduna dayanan modelce 10. ve 11. bölgeler için bulunan boşluk basıncı oranları yapay sinir ağları metodunca bulunan oranlardan çıkarılmıştır. Bulunan hata oranlarının ortalaması alınarak geliştirilen modelin ortalama hatası hesaplanmış ve sonuç olarak modelin başarı oranı bulunmuştur. Bulunan her iki başarı oranının kıyaslanması ile literatürce kabul edilen hata oranlarının içinde kalındığı ve modelin tutarlı olduğu görülmüştür.

Çalışmanın ana amacı olarak şunlar söylenebilir. Zemin profiline ait bir boyutlu, iki boyutlu ve üç boyutlu sonlu farklar metoduna bağlı sıvılaşma potansiyelinin tahmini için model geliştirilmesi ve ayrıca yapay sinir ağları yöntemine dayalı model geliştirilmesi gerçekleştirilmiştir. Modellemelerde %82'lik bir başarı oranı görülmüştür.

Geliştirilen bu yapay sinir ağlarına bağlı modelin performansının artırılması, sonlu farklar yöntemi tabanlı model sonuçları ve sinir ağı algoritması tabanlı model sonuçları ve basitleştirilmiş yöntem sonuçlarının karşılaştırılması yapılmıştır. Bir boyutlu, iki boyutlu ve üç boyutlu sonlu farklar metodundaki modellerin sonuçlarının karşılaştırılması, böylelikle farklı boyutlarda sıvılaşma analizleri yapıldığında gerek arazinin en iyi şekilde modellenebilmesi gerekse de analizler sonucunda hangi boyutlandırma ile bölgenin sıvılaşma riskini en doğru şekilde tahmin edilebileceğinin saptanması hedeflenmiştir.

Son olarak sıvılaşma potansiyelinde deprem parametresi (deprem etkisi) sabit tutularak diğer etkin parametreler olan zemine bağlı parametrelerin yapay sinir ağları modelince etkinlik yüzdelerinin hesaplanarak karşılaştırılması yapılmış ve sıvılaşma oluşumunu etkileyen bir çok faktörden hangilerinin araştırma alanı olan Kurbağlıdere bölgesinde daha etkin olduğunun saptanması amaçlanmıştır. Bu anlamda, zemin içsel

sürtünme açısı ve SPT-N değerlerin zemin sıvılaşmasında en etkin parametreler olduğu saptanmıştır.

Zeminlerin sıvılaşma potansiyelinin tahmininde kullanılan bu farklı metotların karşılaştırılması için geliştirilen bu iki model ile zeminlerde sıvılaşma potansiyeli olasılıklarında kullanılacak ve uygulamanın başka araştırmalar ve çalışmalarda tahmin aracı olarak kullanılmasını sağlayan başarılı sonuçlara varılmıştır.

1. INTRODUCTION

Soil liquefaction is often one of the major causes of damage to buildings, highways, bridges and other infrastructure components in an earthquake. Any improvement to the existing methods for assessing liquefaction potential is a contribution to the field of civil engineering.

In recent years, performance based design concepts in earthquake engineering including finite difference method based models have been receiving wide acceptance. Today, artificial intelligence method approaches are new valid methods for evaluating liquefaction potential.

1.1 Background

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by strong ground motion or earthquake shaking or other rapid cyclic loading. Liquefaction has historically been responsible for tremendous amounts of damage including landslides, differential settlements, lateral spreading, structural and earth system failures throughout the world. Because of this, liquefaction problems have received a great deal of attention, and great efforts have been made to understand this basic mechanism and phenomenon.

Since the effect of this phenomenon on human life and the economy is significant, geotechnical engineers have given their interest to investigate this problem. The two basic approaches that have been achieved to assess liquefaction potential in layered soils are; i) Laboratory tests based on cyclic stress and strain comparisons of soil samples. Laboratory based techniques are believed to yield less reliable results due to both disturbance of the specimens, and also requires high costs. ii) Empirical methods based on observations from previous earthquakes. Currently, liquefaction engineering procedures profoundly rely on empirical correlations such as cyclic stress ratio for stress based methodology and threshold acceleration for strain based methodology.

Accordingly, parameters derived from field tests are used directly through empirical calculations. These correlations derived from case histories yield reliable results.

1.2 Purpose of the Thesis

Due to the complexity of liquefaction problems, such as nonlinearity of responses, sudden phase transition from solid to liquid behavior, material instability, interaction and relative movement between the porous soil skeleton and interstitial water, limitations on experimental and numerical techniques, material model formulation and so on reliable and accurate predictive methods have yet to be developed.

The scope of this research is limited to analysis of soil liquefaction that utilizes SPT based methods. In this study, the aim is to compare different methods used in evaluating liquefaction potential and their results and also to find the parameters effects and which parameter is the important factor for liquefaction potentials of soils during earthquakes. These methodologies, which are given below are used for generating the specific objectives of the research. These are to develop finite difference model in 3D to predict liquefaction potential of site, develop neural network model to predict liquefaction potential of site, to enhance performance of artificial neural network model of liquefaction analysis, to use neural network model for forecasting studies of liquefaction potentials, to compare simplified methods results with finite difference method based model results and neural network algorithm based model results, to compare 1D, 2D and 3D finite difference method based model results, and also to calculate the importance of the earthquake magnitude by comparing the other parameters that affect the liquefaction potential.

1.3 Methodology

This study is mainly based on numerical models. The first model is based on finite difference method and it is an approximate method for determining derivatives of a function. Depending upon circumstances, the finite difference method may give exact results.

The second model is based on artificial neural network methods and these are systems and computational devices that are constructed to make use of some organizational principles resembling those of the human brain. Normally there are a

large number of highly connected computational nodes (neurons) that are operated and configured in parallel regular architectures. Like human brain, an artificial neural network has the ability to learn; recall and generalize from the data, which are used to train the system and forecast for results.

1.4 Thesis Organization

This thesis is organized as follows:

Chapter 2: Literature Review: This chapter presents a brief discussion on the existing simplified methods proposed by different researchers for assessing soil liquefaction potential and general information about the software's used for liquefaction analysis in past. In addition, the chapter presents an extended literature on neural network modeling (with Neuroshell 2) and on finite difference modeling (with Flac 3D).

Chapter 3: Case Study: This chapter presents the data sets and their laboratory parameters with a summary of soil and earthquake characteristic of chosen Kurbağlıdere, Istanbul, Turkey case study area.

Chapter 4: Development of Models: This chapter introduces finite difference model as a model to predict the liquefaction occurrence with defined model parameters. It also presents the artificial neural network model as a model to use it for forecasting of liquefaction potential of site with model architecture parameters.

Chapter 5: Results: This chapter presents the results of this study.

Chapter 6: Conclusions and Recommendations: This chapter highlights the contribution of this research and suggests recommendations for future works.

2. LITERATURE REVIEW

2.1 Introduction

Liquefaction is one of the most important, interesting, and complex topics in geotechnical earthquake engineering. Liquefaction induced failure has been and continues to be a major cause of damage during earthquakes.

It has also vital importance on the serviceability of bridges, railway embankments, airport runways, submerged tunnels, etc. Therefore, it is important to predict the liquefaction potential and to take precautions against liquefaction in susceptible sites. There are many examples of serious damage caused by liquefaction in earthquakes such as the sudden loss of bearing capacity of foundations, dislocation of retaining walls and the excessive lateral soil movement.

Since the effect of this liquefaction phenomenon on human life and the economy is significant, civil engineers have large interest to investigate this problem.

2.2 Literature Review on Definition of Liquefaction

2.2.1 Definition of liquefaction

As it was originally derived from the Latin verb “liquefacere”, which means to weaken, to melt, or to dissolve, liquefaction is an engineering phenomenon referring to total and sharp loss of soil shear strength due to rapid pore water pressure build up [1]. This term has been first used by Terzaghi and Peck in 1948 to describe the significant loss of strength of very loose sands causing flow failures due to slight disturbance [2].

Similarly, Mogami and Kubo in 1953 used the same term to define shear strength loss due to seismically induced cyclic loading. However, its importance has not been fully understood until 1964 Niigata earthquake, during which the significant causes of structural damage were reported to be due to tilting and sinking of the buildings founded on saturated sandy soils with significant soil liquefaction potential [3].

In 1997 Robertson and Wride reported that as an engineering term, “liquefaction” has been used to define two mainly related but different soil responses during earthquakes; flow liquefaction and cyclic softening. Since both mechanisms can lead to quite similar consequences, it is difficult to distinguish [4].

The term liquefaction has historically been used in conjunction with a variety of phenomena that involve soil deformations caused by monotonic or repeated disturbance of saturated cohesionless soils under undrained conditions. The generation of excess pore pressure under undrained loading conditions is a hallmark of all liquefaction phenomena. When a soil shown in Figure 2.1a is experiencing large effective stress, the intensity of vibration or the number of particular stress cycles must be large to cause slip among grain contacts (Figure 2.1b), cause instability of contacts, and transfer the forces carried by contacts to water and hence increase pore water pressure (Figure 2.1c) [5].

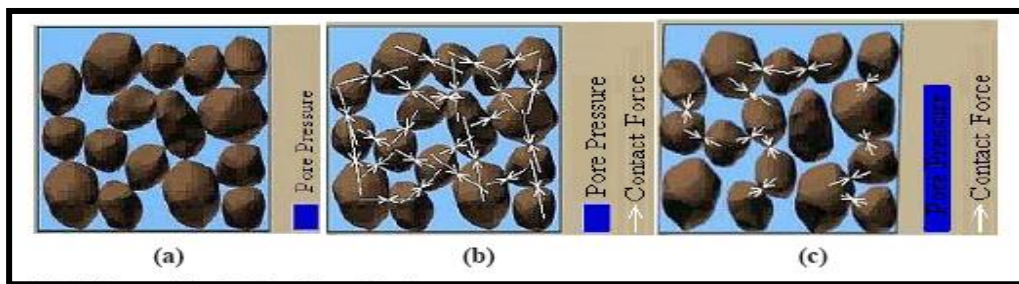


Figure 2.1 : Liquefaction phenomena; (a) soil particles before liquefaction (b) contact forces between individual soil grains (c) soil particles after liquefaction [6].

It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. It is also a phenomenon in which soil loses a large percentage of its shear strength due to the generation of excess of pore water pressure under either monotonic or dynamic loading conditions. In granular soils, liquefaction takes place when the excess pore water pressure is high enough to overcome the contact force between the soil particles (Figure 2.2) [2, 7, and 8].

Therefore, the effective shear strength is reduced due to loss of soil particle contact. When soil loses most of its contact, force the behaviour changes from that of solid to liquid. This causes a softening and weakening of the soil, which ultimately leads to liquefaction. The liquefaction phenomenon is usually associated with undrained loading conditions in which excess pore pressure does not dissipate during shearing [2, 7, and 8].

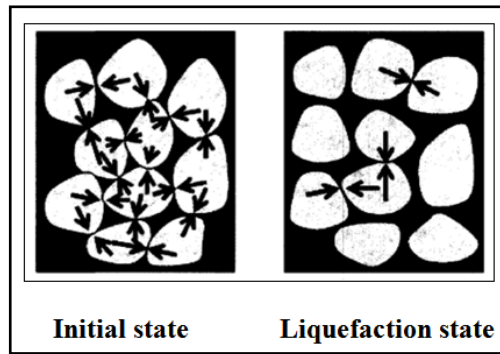


Figure 2.2 : Soil loses its contact force with increasing of pore water pressure and transforms to the liquefaction state [2, 7, and 8].

In other words, liquefaction is defined as “the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress” [9].

It is also the loss of strength and stiffness of saturated loose sand caused by pore water pressure buildup as the sand contracts when shaken [10].

It occurs in saturated soils, that is, soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure is relatively low. However, earthquake shaking can cause the water pressure to increase to the point where the soil particles can readily move with respect to each other. Earthquake shaking often triggers this increase in water pressure, but construction related activities such as blasting could also cause an increase in water pressure. Because liquefaction only occurs in saturated soil, its effects are most commonly observed in low lying areas near bodies of water such as rivers, lakes, bays, and oceans. The effects of liquefaction may include major sliding of soil. Liquefaction leads to ground failures such as sand boils, lateral spreads, flow failures, excessive settlements, bearing capacity failures, pile failures (Figure 2.3a) and overturning of structures (Figure 2.3b) [6].

Liquefaction phenomena could be divided into two main categories; these are the flow liquefaction and cyclic mobility. Flow liquefaction occurs when the shear stress required for a static equilibrium is greater than the shear strength of the soil in its liquefied state. In such a state, the large deformations produced by flow liquefaction actually occur under the effect of gravity or static shear, even though cyclic stresses are responsible for deteriorating the soil strength to a value lesser than the static

shear. The soil spreads out until the shear stresses acting within the mass become so small that they are compatible with its reduced shear strength. Flow liquefaction failures are characterized by sudden nature of their origin, the speed with which they develop, and the large distance over which liquefied soil generally moves [11].

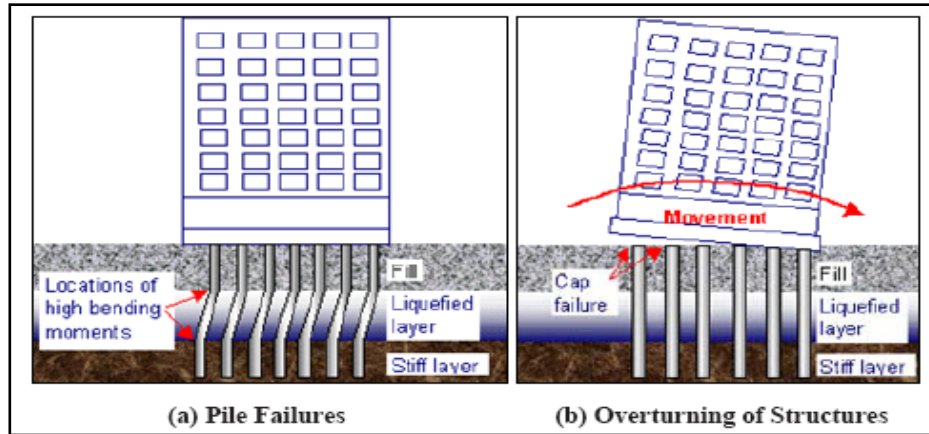


Figure 2.3 : Examples of liquefaction damage [6].

The tendency of dense saturated soils to gradually soften during undrained cyclic loading, but attain limited strains during subsequent static loading is defined as cyclic mobility. Soils subjected to cyclic mobility would first soften but later on stiffen when monotonically loaded under undrained conditions as the tendency to dilate reduces the pore pressures. Cyclic mobility can be differentiated from liquefaction by the fact that liquefied soil does not show any appreciable increase in the stiffness irrespective of the magnitude of displacements [12].

In short, flow liquefaction can occur when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state. In contrast to flow liquefaction, cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil [13, 14].

If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume. If drainage is unable to occur, the tendency to decrease in volume will cause the soil particles to squeeze the water between them, resulting in an increase in pore water pressure. According to the principle of effective stress states, effective stress equals to total stress minus the pore water pressure (2.1) [15].

$$\sigma' = \sigma - u \quad (2.1)$$

where σ' = effective stress, σ = total stress, u = pore water pressure. If the value of σ remains constant, as the pore water pressure u gradually increases, the effective stress σ' is gradually decreased. If the pore water pressure builds up to the point at which it is equal to the total stress, the effective stress becomes zero. As the stiffness and strength of a soil depends on the magnitude of effective stress, the soil loses its strength completely when the effective stress becomes zero and the soil is in a liquefaction state [15].

Evaluation of the potential for liquefaction to occur is accomplished by comparing equivalent measures of earthquake loading and liquefaction resistance. The most common approach to characterization of earthquake loading is through the use of cyclic shear stresses. By normalizing the cyclic shear stress amplitude by the initial effective vertical stress, a cyclic stress ratio (CSR) is obtained. CSR can represent the level of loading induced at different depths in a soil profile by an earthquake. There are different procedures for evaluating the cyclic shear stresses - site response analyses may be performed or a "simplified" approach may be used to estimate CSR as a function of peak ground surface acceleration amplitude [16, 17].

Liquefaction resistance is most commonly characterized on the basis of observed field performance. Detailed investigation of actual earthquake case histories has allowed determination of the combinations of in-situ properties (usually SPT or CPT resistance) and CSR for each case history.

By plotting the CSR with SPT corrected values ($\text{CSR}-(N_1)_{60}$) or with CPT values ($\text{CSR}-q_c$) pairs for cases in which liquefaction was and was not been observed, a curve that bounds the conditions at which liquefaction has historically been observed can be drawn (Figure 2.4) [16, 17].

This curve, when interpreted as the maximum CSR for which liquefaction of a soil with a given penetration resistance can resist liquefaction, can be thought of as a curve of cyclic resistance ratio (CRR). Then, the potential for liquefaction can be evaluated by comparing the earthquake loading (CSR) with the liquefaction resistance (CRR) and this is usually expressed as a factor of safety against liquefaction (2.2),

$$\text{FS} = \text{CRR} / \text{CSR} > 1 \quad (2.2)$$

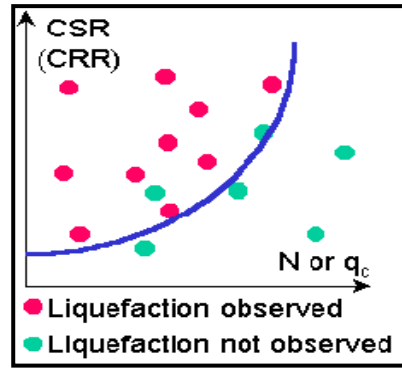


Figure 2.4 : Liquefaction occurrence with CRR (CSR) to SPT-N (q_c) graph [6].

A factor of safety greater than one indicates that liquefaction resistance exceeds the earthquake loading, and therefore that liquefaction would not be expected. There have been many studies focused on identifying the factors and mechanisms causing liquefaction. Size, shape and gradation spectrum of soil particles, initial relative density, fines content, stress level, drainage characteristics, previous strain history, vibration characteristics, period of loading, and tapped air are the main factors affect the liquefaction process [13, 14].

2.2.2 Potentially liquefiable soils

The first step towards the analysis and assessment of liquefaction is the determination of potentially liquefiable soil layers at a given site. For this purpose, the Chinese criteria have been widely used in the engineering practice for longer than two decades. These criteria were modified and formulated, as shown in Table 2.1 [18]. Later, further modifications were applied to these criteria by Andrews and Martin in 2000 for silt and clay definitions, as shown in Table 2.2 [19].

Table 2.1 : Formulated Chinese criteria proposed by Seed and Idriss [18].

Potentially Liquefiable Soils	
Fines Content (<0.005 mm)	$\leq 15\%$
Liquid Limit (LL)	$\leq 35\%$
Water Content (W)	$\geq (0.9 \times LL)\%$

Seed et al. (2003) directly related the building penetration case histories that occurred during the 1999 earthquake events in Kocaeli, Turkey and Chi Chi, Taiwan to soil liquefaction, and recommended new criteria replacing the Chinese criteria, as shown in Figure 2.5. These criteria classify all soils with a plastic index (PI) less than 12

and liquid limit (LL) less than 37 as potentially liquefiable, provided that the soil natural moisture content is greater than $(0.8 \cdot LL)$ [9].

Table 2.2 : Modified Chinese criteria proposed by Andrews and Martin [19].

	Liquid Limit <32%	Liquid Limit $\geq$ 32%
Clay Content (<0.002 mm) <10%	Potentially Liquefiable	Further studies required considering plastic non-clay sized grains
Clay Content (<0.002 mm) $\geq$ 10%	Further studies required considering non-plastic clay sized grains	Non-Liquefiable

The most recent attempt was done by Boulanger and Idriss in 2004 [21]. They evaluated liquefaction resistance and deformation of granular and fine grained soils, using cyclic laboratory tests, and recommended the new criteria shown in Figure 2.6, which are also a replacement to the Chinese criteria [9].

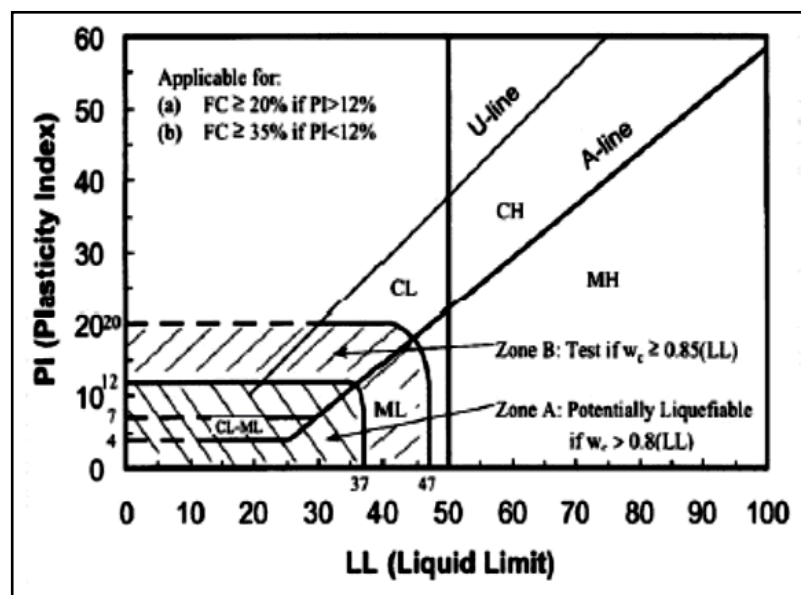


Figure 2.5 : Criteria for liquefaction susceptibility of fine-grained sediments proposed by Seed [20].

This methodology divides the deformation behavior of fine grained soils into two different types; “Sand-Like” and “Clay-Like”, where soils within the sand-like behavior region are susceptible to liquefaction and have substantially lower values of cyclic resistance ratio, CRR, than those within the clay-like behavior region [9].

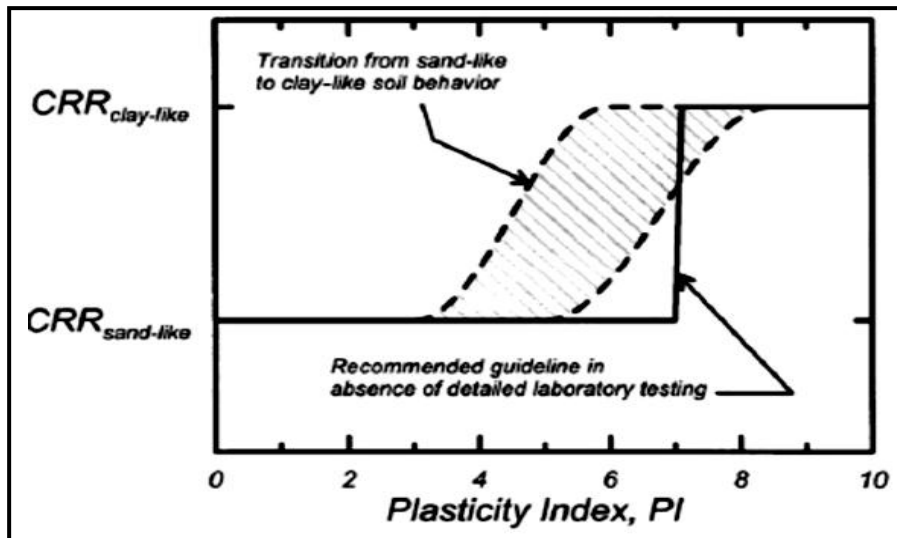


Figure 2.6 : Criteria for differentiating between sand-like and clay-like sediment behavior proposed by Boulanger and Idriss [21].

2.2.3 Liquefaction susceptibility

Not all soils are susceptible to liquefaction, even under severe earthquake ground motion. There are some basic requirements for the occurrence of liquefaction. In 1996 Kramer provided four general categories of susceptibility criteria by which liquefaction susceptibility can be judged; historical, geologic, compositional, and state. Historical criteria are first to recognize when determining if a soil is susceptible to liquefaction. Based on post-earthquake investigations and case histories, research studies show that a site that has liquefied in past can suffer liquefaction again when soil and groundwater conditions have remain unchanged [22].

The similarities of soil and water conditions concluded from the liquefied sites can be further applied as a baseline database to check other sites prone to liquefaction. A map of liquefaction susceptibility then can be constructed based on the number of observed historical evidences of liquefaction. Geologic criteria such as depositional environment, age of the soil and groundwater conditions can affect the liquefaction susceptibility of soils. The range of liquefiable soil deposits is a relatively narrow range of geological environments. In general, soil deposited in loose states, like fluvial and colluvial deposits have high liquefaction susceptibility when saturated. As mentioned previously, liquefaction occurs in saturated soils, so the position of the groundwater table also affects of soil liquefaction susceptibility. Compositional criteria recognize the fact that the characteristics the influence of the volume change behavior in soils is responsible to liquefaction susceptibility. Many compositional

factors, such as soil gradation, particle shape, fines content, plasticity index, liquid limit, and water content (w_c), influence soil's liquefaction susceptibility [23].

In early liquefaction potential research, Wang in 1979 first established criteria for liquefaction susceptibility based on data from sites where liquefaction was or was not observed after earthquakes in China. Based on these data, in 1982 Seed and Idriss proposed three criteria for clayey soils to determine if the soil is susceptible to liquefaction. Only soils satisfying all three criteria were considered to be susceptible to liquefaction [23].

State criteria are the final factors used to evaluate soil liquefaction susceptibility. A soil still may not be susceptible to liquefaction if it cannot satisfy the state criteria; even it meets all three previous criteria. The term “state” refers to the soil's initial effective stress and density at the time of the earthquake. Because of the relationship between excess pore pressure, soil density, and initial stress conditions, liquefaction susceptibility depends strongly on the initial state of the soil. State criteria, unlike the criteria mentioned before, are different for flow liquefaction and cyclic mobility. To illustrate the role of initial states in evaluating liquefaction susceptibility, two basic concepts of cohesionless soil behavior are introduced as requirement [24].

Liquefaction susceptibility evaluation is also based on another criteria. From a series drained, strain controlled triaxial tests of loose and initially dense sand specimens, Casagrande in 1936 showed that at large strains, all specimens approached the same density and continued to shear with constant shearing resistance. The void ratio corresponding to this constant density was termed the critical void ratio, by performing tests at different confining pressures, Casagrande constructed a line called critical void ratio, CVR, line, as shown in Figure 2.7. This CVR line is used as a boundary to separate loose and dense states under monotonic loading. Since positive excess pore pressure are generated when soil behaves contractively, saturated soils with initial void ratios that plot above the CVR line are susceptible to flow liquefaction. The steady state of deformation was first proposed based on a series stress controlled undrained triaxial tests. A line, called the steady state line (SSL), represents the relationship between void ratio and effective confining pressure in the steady state of deformation [24].

In practical, the SSL is useful for identifying the conditions under which soil may or may not be susceptible to flow liquefaction. Figure 2.7 shows that a soil whose state lies above the SSL is susceptible to flow liquefaction, but only if the static shear stress exceeds its steady state strength. The two concepts, CVR and SSL are particularly useful for understanding the susceptibility to flow liquefaction. Based on field observation and laboratory testing results, liquefaction characteristics of cohesionless soils are affected by a number of factors. These factors affecting liquefaction susceptibility are given below.

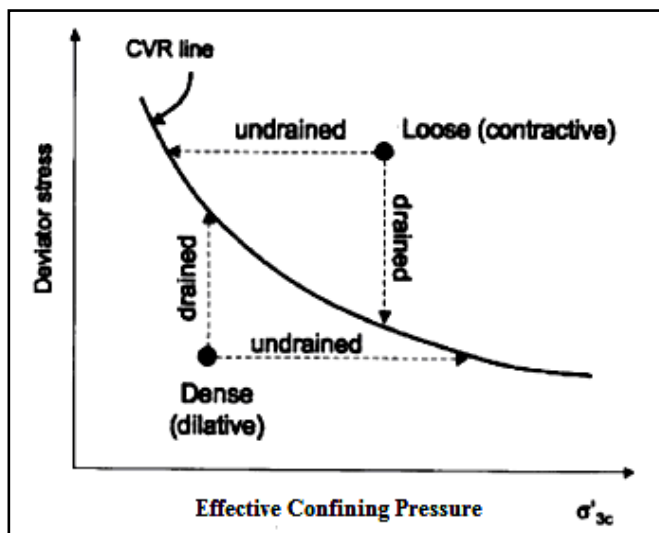


Figure 2.7 : CVR line usage as a boundary between loose and dense states [25].

2.2.3.1 Grain size distribution and soil types

The type of soil most susceptible to liquefaction is one in which the resistance to deformation is mobilized by friction between particles. If other factors such as grain shape, uniformity coefficient and relative density are equal, the frictional resistance of cohesionless soil decreases as the grain size of soils becomes smaller. In 1970 Tsuchida summarized the results of sieve analyses performed on a number of alluvial and diluvial soils that were known to have liquefied or not to have liquefied during earthquakes; and proposed ranges of grain size curves separating liquefiable and non-liquefiable soils.

A soil with a gradation curve falling in the zones between the outer and inner curves is less likely to liquefy. Soils with a higher percentage of gravels tend to mobilize higher strength during shearing, and to dissipate excess pore pressures more rapidly than sands.

However, there are case histories indicating that liquefaction has occurred in loose gravelly soils during severe ground shaking or when the gravel layer is confined by an impervious layer. The space between the two curves farthest to the left reflects the influence of fines in decreasing the tendency of sands to densify during seismic shearing. Fines with cohesion and cementation tend to make sand particles more difficult to liquefy or to seek denser arrangements. However, non-plastic fines such as rock flour, silt and tailing slimes may not have as much of this restraining effect. In 1985 Ishihara stated that clay or silt size materials having a low plasticity index value will exhibit physical characteristics resembling those of cohesionless soils, and thus have a high degree of potential for liquefaction [26].

In 1989 Walker and Steward based on their extensive dynamic tests on silts, have also concluded that non-plastic and low plasticity silts, despite having their grain size distribution curves outside of Tsuchida's boundaries for soils susceptible to liquefaction, have a potential for liquefaction similar to that of sands and that increased plasticity will reduce the level of pore pressure response in silts. This reduction, however, is not significant enough to resist liquefaction for soils with plasticity indices of 5 or less. Even though major slide movements during earthquakes have occurred in clay deposits, they are commonly considered to be non-liquefiable during earthquakes in the sense that an extensive zone of clay soil is converted into a heavy fluid condition.

However, it is believed that quick clays may lose most of their strength after strong shaking and that other types of clay may lose a proportion of their strength resulting in slope failures. Frequently, landslides in clay deposits containing sand or silt lenses are initially triggered by the liquefaction of these lenses before any significant strength loss occurs in the clay. This has been supported by laboratory test results which indicate that the strain required to liquefy sands is considerably smaller than the strain required to overcome the peak strength of cohesive soils.

There is also evidence that uniformly graded materials, generally having a uniformity coefficient smaller than five, are more susceptible to liquefaction than well graded materials and that for uniformly graded soils, fine sands tend to liquefy more easily than coarse sands, gravelly soils, silts or clay.

2.2.3.2 Relative density

Laboratory test results and field case histories indicate that, for a given soil, initial void ratio or relative density is one of the most important factors controlling liquefaction. Liquefaction occurs principally in saturated clean sands and silty sands having a relative density less than 50%.

For dense sands, however, their tendency to dilate during cyclic shearing will generate negative pore water pressures and increase their resistance to shear stress. Lower limit relative density beyond which liquefaction will not occur is about 75%.

2.2.3.3 Earthquake loading characteristics

The vulnerability of any cohesionless soil to liquefaction during an earthquake depends on the magnitude and number of cycles of stresses or strains induced in it by the earthquake shaking. These in turn are related to the intensity, predominant frequency, and duration of ground shaking.

2.2.3.4 Vertical effective stress and over consolidation

It is well known that an increase in the effective vertical stress increases the bearing capacity and shear strength of soil, and thereby increases the shear stress required to cause liquefaction and decreases the potential for liquefaction. From field observations it has been concluded by investigators that saturated sands located deeper than 15 to 18 m (50 to 60 feet) are not likely to liquefy. These depths are in general agreement with Kishida in 1969 who states that a saturated sandy soil is not liquefiable if the value of the effective overburden pressure exceeds 190 kN/m^2 (2 tsf). Both theory and experimental data show that for a given soil a higher over consolidation ratio leads to higher lateral earth pressure at rest and thereby increases the shear stress ratio required to cause liquefaction [27].

2.2.3.5 Age and origin of the soils

Natural deposits of alluvial and fluvial origins generally have soil grains in the state of loose packing. These deposits are young, weak and free from added strength due to cementation and aging. In 1977, Youd and Hoose stated that, as a rule of thumb, alluvial deposits older than late [27].

Pleistocene (10,000- 130,000 years) are unlikely to liquefy except under severe earthquake loading conditions, while late Holocene deposits (1,000 years or less) are most likely to liquefy, and earlier Holocene (1,000-10,000 years) deposits are moderately liquefiable.

2.2.3.6 Seismic strain history

It has been demonstrated from laboratory test results that prior seismic strain history can significantly affect the resistance of soils to liquefaction [27].

Low levels of prior seismic strain history, as a result of a series of previous shakings producing low levels of excess pore pressure, can significantly increase soil resistance to pore pressure buildup during subsequent cyclic loading. This increased resistance may result from uniform densification of the soil or from better interlocking of the particles in the original structure due to elimination of small local instabilities at the contact points without any general structural rearrangement taking place [27]. Large strains, however, associated with large pore pressure generation and conditions of full liquefaction can develop weak zones in the soil due to uneven densification and redistribution of water content and thus lower the resistance of the soil to pore pressure generation during subsequent cyclic loading [28].

2.2.3.7 Degree of saturation

Liquefaction will not occur in dry soils. Only settlement, as a result of densification during shaking, may be of some concern. Available laboratory test results show liquefaction resistance for soils increases with decreasing degree of saturation, and that sand samples with low degree of saturation can become liquefied only under severe and long duration of earthquake shaking [27].

2.2.3.8 Thickness of sand layer

In order to induce extensive damage at level ground surface from liquefaction, the liquefied soil layer must be thick enough so that the resulting uplift pressure and amount of water expelled from the liquefied layer can result in ground rupture such as sand boiling and fissuring. If the liquefied sand layer is thin and buried within a soil profile, the presence of a non-liquefiable surface layer may prevent the effects of the at depth liquefaction from reaching the surface.

Ishihara in 1985 has set up a criterion to stipulate a threshold value for the thickness of a non-liquefiable surface layer to avoid ground damage due to liquefaction. It should also be noted that even though the thickness of a non-liquefiable surface layer exceeds the threshold thickness, the ground surface may still experience some settlement which may be undesirable for certain settlement sensitive structures [26].

2.2.4 Effects of liquefaction on soils

If a soil is susceptible to liquefaction, and state of liquefaction is initiated and triggered, the remaining problem is to estimate the expected effects of liquefaction.

Some major seismic hazards induced by soil liquefaction include flow sliding, lateral spreading displacements, and ground surface settlement. Descriptions and discussion of these major effects of liquefaction will be outlined as following. These effects which include flow sliding, lateral spreading and ground surface settlement can be grouped into main categories.

2.2.4.1 Liquefaction – Induced instability relationship

In the group of liquefaction - induced instability, flow sliding and lateral spreading are the two dominant mechanisms of instability. Flow slides usually cause serious damage when they occur, so the potential for triggering of a flow slide becomes important for engineers. Occurrence of a flow slide is affected by several factors; soil properties, the slope profile and loadings induced by an earthquake. Because of the significant consequence of flow sliding, engineers are more interested to know if a flow slide will occur, that is, if the factor of safety for flow sliding is less than unity. The potential for flow sliding is based on the static driving and the available resisting stress following the initiation of liquefaction. The shear resistance of the soil is reduced by the generation of pore pressure during cyclic loading. When the soil strength drops to a point at which it becomes smaller than the driving stress, flow sliding occurs. The available shear strength at this condition is defined as the residual shear strength of the liquefied soil. Lateral spreading is another type of instability induced by soil liquefaction. Liquefaction induced lateral spreading displacements are usually observed along river banks, beneath coastal embankments, and at bridge abutments. Compared with flow slides, lateral spreading displacement is phenomenon of smaller scale in size but occurs much more frequently than flow sliding [29].

The most commonly used evaluation methods for liquefaction induced lateral spreading are empirical in nature, in which earthquake loading is described by source parameters, magnitude (M) and source to site distance (R), rather than ground motion parameters. These prediction equations estimate lateral spreading displacements based on site profile, soil properties and combinations of M and R [30].

As mentioned before, the most serious effect of soil liquefaction could be the occurrence of flow slides.

There is often little or no warning that a flow failure will occur because the reduction of soil strength occurs suddenly, once the effective stress path of a soil reaches the flow liquefaction surface (FSL). All such failures occur on sloping ground and they can occur even on gently sloping ground if the residual strength of the liquefied soil is very low. Once a flow failure initiates, large masses of fluid like soil can move down slope at velocities approaching several meters per second. Such flows can destroy structures in their paths and deposits huge volumes of soil once equilibrium has been reached [34].

Many investigators had devoted a lot effort to the study of post-liquefaction shear strength. This characteristic controls the potential for flow sliding or flow failure. Because the post liquefaction shear strength, or residual shear strength, can be very low, such that the static shear stresses can drive permanent deformations, which can become very large as in cases of flow sliding. Because of the difficulties encountered in practical application of laboratory based procedures for steady state strength, Seed in 1987 proposed a case history based method to address the problem. Seed used penetration resistance, most commonly SPT resistance, as the measure of in-situ density and related it to the residual strength (S_r) [34].

As mentioned before, the second effect is the lateral spreading and it refers to the uni-directional movement of liquefied soil due to the presence of small static shear stresses that exist prior to earthquake shaking in the soil. Lateral spreading can occur on gently sloping ground, or near a free face, and can be a major engineering concern because infrastructure is frequently located in these areas. Lateral spreading has caused significant damage to infrastructures over the past several decades. Several studies have been conducted by researchers during that time to better understand the mechanics of lateral spreading and model its occurrence [31].

2.2.4.2 Liquefaction – Induced settlement relationship

Post liquefaction ground surface settlement is associated with the dissipation of excess pore water pressure generated during seismic loading. After soil liquefaction, a soil mass typically consolidates, resulting in volume change that is manifested in the form of ground surface settlement. In urban areas, liquefaction induced ground surface settlements can cause serious damage to buildings and other structures.

Laboratory tests and case histories have both been used to establish predictive models for post liquefaction ground surface settlement. In predictive models, settlement due to liquefaction is represented by volumetric strain (ϵ_v) [29].

For evaluating the ground surface settlement, we can use different types of analysis styles. These are the probabilistic and the performance based analysis. Both of the two type's use the volumetric strain parameter, so we can say that liquefaction induced settlement relationship is up to the volumetric strain parameter. Earthquake induced excess pore pressure in a liquefied soil dissipates after the seismic shaking. The dissipation of excess pore pressure results in grain re-arrangement in the soil mass as soil particles move into a denser configuration. The volumetric strain resulting from soil re-consolidation causes ground settlement. Such settlement can be destructive to concrete or asphalt pavement, to lifelines buried at shallow depths, particularly if they are connected to a pile supported structure, and to structures supported by shallow foundation. The threat of ground surface settlement is serious to engineers. Based on the observed case histories several researchers' proposed empirical methods to predict or evaluate the potential for post liquefaction ground surface settlement. In these proposed evaluation procedures, the volumetric strain ϵ_v is related to CSR and $N_{1,60}$ and then used for with computation of settlement. A general procedure of evaluation for post liquefaction ground surface settlement method is described in the following paragraph [32].

In 2004 Wu and Seed reviewed previous empirical models of post liquefaction ground surface settlement and performed a series of laboratory tests. Firstly, calculate the cyclic stress ratio, then calculate the value of clean sand $(N_1)_{60,CS}$. After that, find the volumetric strain by using SPT – CSR figure and finally, calculate the settlement by integrating the volumetric strain, over the thickness of the liquefied soil layers [32].

2.2.4.3 Liquefaction – Earthquake relationship

If one discusses the modification of ground motions, one can say that when soil suffers liquefaction during an earthquake, its stiffness decreases significantly, which can strongly influence the amplitude and frequency content of the motion of the soil above the liquefied zone. The changed properties of the soil tend to filter out the high frequency ground motions, thus only allowing lower frequency waves to continue to the ground surface.

The lower frequency ground motions can result in large, rolling displacements which can be especially destructive to soft structures with low natural frequencies. However, dilation associated with phase transformation behavior can cause high-frequency spikes following the initiation of liquefaction.

Youd and Carter in 2003 examined ground motion records from five instrumented sites in order to analyze the effects of soil softening caused by liquefaction during cyclic loadings. The softening of the liquefied soil is due to the generation of pore pressure which causes reduction of effective stress and stiffness of liquefied soil [33].

Because of the importance of the earthquake – liquefaction relationship, Seed and Idriss in 1982, introduced correlation for different magnitude earthquake, termed magnitude scaling factors (MSF), to extend the simplified curve for $M_w=7,5$ to other magnitude earthquakes. These factors are used to scale the simplified curves upward and downward by the following equation (2.3).

$$CSR = CRR_{7,5} \times MSF \quad (2.3)$$

where $CRR_{7,5}$ = Cyclic resistance ratio, MSF = Magnitude scaling factor [18].

2.2.4.4 Flow liquefaction surface and excess pore pressure relationship

The stress condition at which flow liquefaction occurs is most easily understood with the aid of a stress path. When plotted in the stress path space, the effective stress conditions at the initiation of liquefaction form a three dimension surface that is referred to as the flow liquefaction surface (FLS) [35].

Based on the definition of the FLS, if the stress conditions of an element of soil are such that its stress path reaches the FLS under undrained conditions, flow liquefaction will be triggered and the shearing resistance will be reduced to the steady state strength. Therefore, the FLS describes the effective stress conditions at which flow liquefaction is initiated. The FLS can be applied to both cyclic and monotonic loading. The development of flow liquefaction follows two stages. The first stage involves the generation of excess pore pressure to move the stress path from its initial state to the FLS. The second stage involves strain-softening at which the large strains develop when the effective stress path moves from the FLS to the steady state point [36].

Different liquefaction phenomena require different levels of pore pressure to occur. As mentioned before, flow liquefaction can occur when the shear stress for static equilibrium is greater than the steady state strength. Hence the initial states plotted in the shaded zone of Figure 2.8 are vulnerable to flow liquefaction. Very small static or dynamic disturbance can trigger flow liquefaction if the soil's initial state is near the FLS [37].

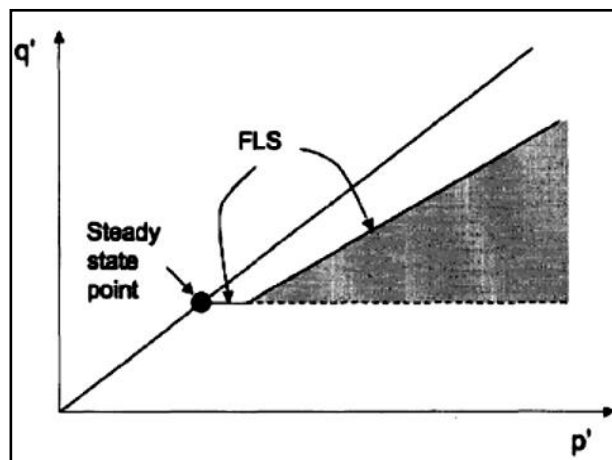


Figure 2.8 : Zone of initial stress conditions susceptible to flow liquefaction [36].

In contrast, the soil has less chance to trigger flow liquefaction if its initial stress state is further from the FLS, because a higher level of excess pore pressure is required. Cyclic mobility is a different story from the standpoint of excess pore pressure. The initial states of a soil located in the shaded area of Figure 2.9 indicate the possible occurrence of cyclic mobility [36].

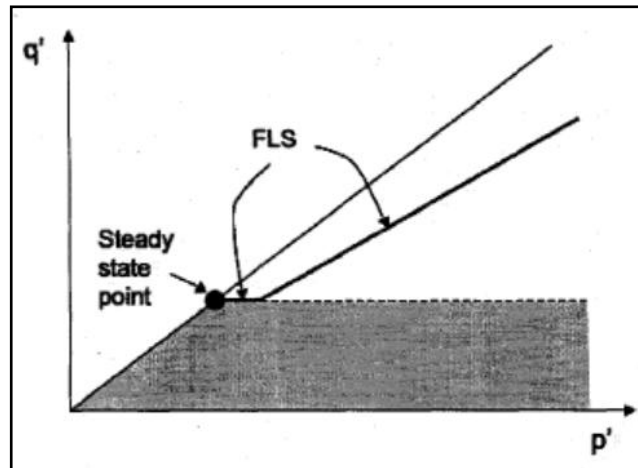


Figure 2.9 : The susceptible area of cyclic mobility [36].

2.2.5 Evaluation of initiation of liquefaction

Several methods are used in contemporary engineering practice for evaluating the potential of liquefaction's initiation. Most available methods can be divided into two common categories: the cyclic stress and cyclic strain methods. In each category, the characteristics of earthquake loading and the available liquefaction resistance of soils control the initiation of liquefaction [36]

Note that the cyclic mobility can occur in both loose and dense soils. There are three combinations of initial conditions and cyclic loading condition for cyclic mobility occurrence. Each of these combinations represents different requirements of changes in effective stress states and these are for (a) no stress reversal and no exceedance of the steady state strength; for (b) no stress reversal with momentary period of steady-state strength exceedance; and for (c) stress reversal with no exceedance of steady-state strength, [36, 38]

2.2.5.1 Cyclic stress approach

The cyclic stress approach uses cyclic shear stress to characterize both the loading and resistance of the soil. The earthquake induced cyclic shear stress is compared with cyclic shear stress required to cause liquefaction of the soil. Both loading and resistance stresses are therefore expressed in terms of cyclic shear stresses. At sites where the loading exceeds the resistance, liquefaction is expected to occur.

Cyclic mobility usually occurs in gently sloped and flat grounds adjacent to bodies of water. Such soils have very low to no initial shear stresses and fall under the flow liquefaction zone. With low static shear stresses, when subjected to dynamic loading, the soil would experience cyclic displacements but does not flow since the shear stresses applied on the soil are lower than the steady state strength [36].

Although the cyclic mobility usually results in lower deformations than flow liquefaction, the magnitude of the deformations can still be much higher than any structures constructed on top of it can accommodate. Level ground liquefaction is one special case of cyclic mobility where the soil has no initial shear stresses. The earthquake motion can only generate vertical displacements in the soil causing it to densify. The result is generation of pore pressures throughout the thickness of the liquefiable layer.

This produces upward flow of water to dissipate the excess pore pressures, and thus resulting in additional decrease in the effective stresses. The final result is excessive vertical displacements on the surface, flooding of lower surface elevation areas due to the surface water from upward flow and the formation of sand boils [39].

During earthquakes, the excess pore pressure is related to the amplitude and duration of the shaking, which are reflected in the time history of cyclic shear stresses. Thus the seismic loading can be expressed in terms of cyclic shear stresses. There are two ways to derive the loading conditions: by a detailed ground response analysis or using a simplified approach. Ground response analysis can be used to predict time histories of shear stress at various depths within a soil deposit. To relate the actual, transient earthquake induced loading to the type of loading applied in the laboratory tests in which the cyclic stress stresses have uniform amplitude, a conversion is required. The intent of the conversion is to produce a series of harmonic stress cycles that would cause an increase in pore pressure equivalent to that of the irregular time history [36].

To develop the conversion, in 1975 Seed determined the number of uniform stress cycles, N_{eq} , to represent the earthquake induced loading as a set of shear stress histories with uniform amplitude.

The uniform shear stress level used in Seed's N_{eq} method was at an amplitude of 65% of the peak cyclic shear stress (i.e., $T_{cyc} = 0.65 \cdot T_{max}$). Other stress levels have been proposed by other researchers, but the 65% level is most commonly used nowadays. A simplified procedure for level sites to estimate the uniform cyclic shear stress amplitude was then developed (2.4) [40].

$$\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d \quad (2.4)$$

where a_{max} is the peak ground surface acceleration, g is the acceleration of gravity, σ_v is the total vertical stress, and r_d represents the value of a stress reduction factor at the depth of interest. The uniform cyclic shear stress is assumed to be applied for the equivalent number of cycles [36].

The approximated mean value curve of r_d was expressed as (2.5) [9].

$$r_d = \frac{1.0 - 0.4113 z^{0.5} + 0.04052 z + 0.001753 z^{1.5}}{1.0 - 0.4177 z^{0.5} + 0.05729 z - 0.006205 z^{1.5} + 0.001210 z^2} \quad (2.5)$$

in which z is the depth beneath ground surface in meters. A different r_d equation was recently proposed by Idriss and Boulanger (2006), and can be expressed as (2.6)

$$\ln(r_d) = \alpha(z) + \beta(z)M \quad (2.6)$$

where $\alpha(z)$ and $\beta(z)$ are functions of z , and M is moment magnitude.

Whether using detailed ground response analysis or simplified procedures, the earthquake induced loading is represented by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles. The cyclic shear stress is usually normalized by the initial vertical effective stress to obtain a dimensionless cyclic stress ratio (2.7).

$$CSR = \frac{\tau_{cyc}}{\sigma_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}} r_d \quad (2.7)$$

In 1982 Seed and Idriss introduced correction factors named "magnitude scaling factors" (MSFs) to apply to CSR in order to account for magnitudes smaller or larger than 7.5.

The resulting CSR expression is now commonly used for evaluation of liquefaction initiation [9], and can be evaluated as (2.8).

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} \frac{r_d}{MSF} \quad (2.8)$$

where MSF, the magnitude scaling factors, has different values and closed form expressions proposed by various investigators [18].

The liquefaction resistance of a soil element can be described in terms of the level of loading required to move it from initial state to the failure state. The liquefaction resistance of a soil can be evaluated either by methods of laboratory tests or by in-situ tests and observations of liquefaction behavior in past earthquakes. Laboratory tests have shown that liquefaction resistance is influenced by soil density, cyclic stress amplitude, and the number of loading cycles [36].

After determining the cyclic loading parameters and the liquefaction resistance of soil, evaluation of liquefaction potential can be accomplished by simply comparing the loading and resistance characteristics throughout the soil deposit of interest. Liquefaction can be expected at depths where loading exceeds resistance, when the factor of safety against liquefaction, as expressed below, is less than 1 (2.9) [36].

$$FS_L = \frac{\text{cyclic shear stress required to cause liquefaction}}{\text{equivalent cyclic shear stress induced by earthquake}} = \frac{\tau_{cyc, L}}{\tau_{cyc}} = \frac{CRR}{CSR} \quad (2.9)$$

In the equation, the numerator, cyclic resistance ratio (CRR), meaning the minimum cyclic shear stress to cause liquefaction is actually the resistance the soil has to prevent the occurrence of liquefaction. A graphical example of the evaluation, showing how cyclic shear stress and liquefaction resistance can vary with depth, is illustrated in Figure 2.10. It should be noted that significant excess pore pressure can develop even if the computed factor of safety is greater than 1.0 [36, 38].

Using factor of safety to represent liquefaction potential is a convenient tool and provides a measure that is easy to understand. Based on the widely applied SPT tests and correspondingly large researches, there are two sets of evaluation procedures that are recognized as applicable standards for evaluation of liquefaction potential. (Table 2.3, 2.4) [36].

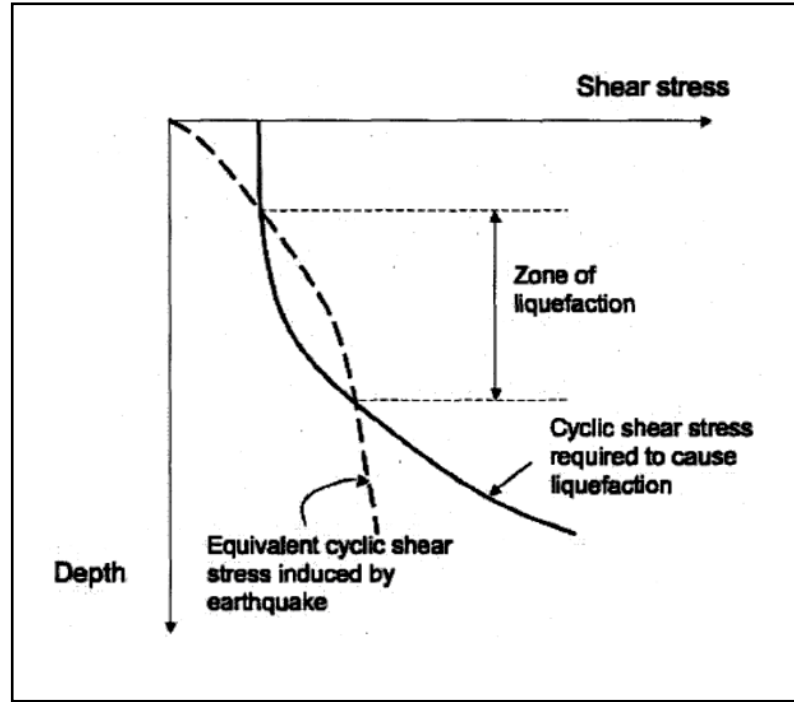


Figure 2.10 : Potential zone of liquefaction [38].

Table 2.3 : Comparison of SPT based evaluation methods for liquefaction initiation - part I [9, 41].

NCEER (Youd et al., 2001)	Idriss and Boulanger (2006)
$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{v0}}{\sigma'} \frac{r_d}{MSF}$	$CRR = CRR_{\sigma=1 \text{ tsf}} K_{\sigma}$ $N_{1,60} = C_N N \frac{ER}{60}$
Depth reduction factor, r_d	
$r_d = \text{Equation 2.10}$	$r_d = \exp(\alpha(z) + \beta(z)M_w)$ where $\alpha(z) = -1.0212 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$ $\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$ with z in meters and limited to a maximum depth 20 m below which the site-specific response analysis is recommended.
Magnitude scaling factor, MSF	
$MSF = \frac{7.5}{MW} 2.95$ for $M_w \leq 7.5$ $MSF = \frac{7.5}{MW} 2.56$ for $M_w > 7.5$	$MSF = \min\left(6.9 \exp\left(\frac{-MW}{4}\right) - 0.058, 1.8\right)$
Normalization factor, C_N	
$C_N = \min\left(\left(\frac{Pa}{\sigma'_{v0}}\right)^{0.5}, 1.7\right)$	$C_N = \min\left(\left(\frac{Pa}{\sigma'_{v0}}\right)^a, 1.7\right)$ Where $a = 0.784 - 0.0768 \sqrt{N_{1,60}}$ and $N_{1,60}$ is limited to a maximum value of 46.

Table 2.4 : Comparison of SPT based evaluation methods for liquefaction initiation - part II [9, 41].

NCEER (Youd et al., 2001)		Idriss&Boulanger (2006)
Calculation of $N_{1,60cs}$		
$N_{1,60cs} = \alpha + \beta N_{1,60cs}$ where $\alpha = \begin{cases} 0 & FC \leq 5\% \\ \exp(1.76 - \frac{190}{FC}) & 5\% < FC < 35\% \\ 1.0 & FC \geq 35\% \end{cases}$ $\beta = \begin{cases} 1.0 & FC \leq 5\% \\ 0.99 + \frac{FC^{1.5}}{1000} & 5\% < FC < 35\% \\ 1.2 & FC \geq 35\% \end{cases}$		$N_{1,60cs} = N_{1,60} + \Delta N_{1,60}$ where $\Delta N_{1,60} = \exp [1.63 + \frac{9.7}{FC+0.1} - (\frac{15.7}{FC+0.1})^2]$ where FC is in percent.
Resistance, $CRR_{\sigma=1\ tsf}$		
$CRR_{\sigma=1\ tsf} = \frac{1}{\frac{34 - N_{1,60cs}}{50} - \frac{1}{200}} + \frac{N_{1,60cs}}{135}$		$CRR_{\sigma=1\ tsf} = \exp(f(N_{1,60cs}))$ $f(N_{1,60cs}) = + (\frac{N_{1,60cs}}{126})^2 - (\frac{N_{1,60cs}}{23.6})^3 + (\frac{N_{1,60cs}}{25.4})^4 - 2.8$
Overburden stress correction, $K\sigma$		
$K\sigma = \min ((\frac{\sigma'_{vo}}{Pa})^{f-1} .1.0)$ where $f = 0.7 \sim 0.8$ for $D_r = 40\% \sim 60\%$ $f = 0.6 \sim 0.8$ for $D_r = 60\% \sim 80\%$		$K\sigma = \min (1 - C\alpha \ln (\frac{\sigma'_{vc}}{Pa})^{f-1} .1.0)$ where $C\alpha = \frac{1}{18.9 - 2.55 \sqrt{N_{1,60cs}}}$ and $N_{1,60cs}$ is limited to a maximum value of 37.

2.2.5.2 Cyclic strain approach

In addition to the cyclic stress approach, a cyclic strain approach has been developed to evaluate liquefaction initiation. Laboratory tests have shown that excess pore pressures generated by cyclic loading are much more strongly correlated to strain amplitude than stress amplitude. The loading conditions in the cyclic strain method are expressed by the concepts of an equivalent series of uniform strain cycles, much like the one used in cyclic stresses approach. In 1982 Dobry proposed a simplified method for estimating amplitude of the uniform cyclic strain (2.10) [36].

$$\gamma_{cyc} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v r_d}{G(\gamma_{cyc})} \quad (2.10)$$

where $G(\gamma_{cyc})$ is the shear modulus of the soil at $\gamma = \gamma_{cyc}$.

The insensitivity of pore pressure generation to the factors (density, soil fabric, OCR, etc.), except the cyclic strain, is the critical benefit of the cyclic strain approach. However, in cyclic strain approach, there is no factor of safety against liquefaction used since both loading and resistance are expressed in terms of strain rather than stresses [36].

2.2.5.3 Probabilistic approach

There are many potential sources of uncertainty in both the loading and resistance aspects of the liquefaction potential problem. Consideration of these requires the development of probabilistic approaches for evaluation of liquefaction potential. Probabilistic methods have been developed using in-situ tests, like SPT, CPT, and V_s . These methods use various statistical classifications and regression analyses to predict probabilities of liquefaction based on different combinations of loading and resistance parameters. To apply the probability analysis from in-situ tests, a well-defined data source of high quality is usually required [36].

2.2.6 In-Situ test methods

The merit of the simplified procedure lies in that it is relatively simple and there are successful engineering applications. Nevertheless, it is largely based on the results of laboratory tests. The unavoidable disturbance during sampling and reconsolidation in laboratory tests limit the application of laboratory results to some extent.

Therefore, characterization of liquefaction potential using laboratory testing is difficult. A simpler method to assess the cyclic resistance of soil is based on results of in situ tests. Commonly used field tests to evaluate liquefaction strength include Standard Penetration Test (SPT), Cone Penetration Test (CPT), Shear Wave Velocity Test (V_s) and Becker Penetration Test (BPT). Primary advantages and disadvantages of each test are summarized by Youd in 2001 as given in Table 2.5 [9].

Table 2.5 : Comparison of various field tests for assessment of liq. pot. [9].

Feature	SPT	CPT	V_s
Past measurements at liq. sites	Abundant	Abundant	Limited
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain
Quality control and repeatability	Poor to good	Very good	Good
Detection of variability of soil deposit	Good for closely spaced tests	Very good	Fair
Soil types in which test is recommend	Non gravel	Non gravel	All
Soil sample retrieved	Yes	No	No
Test measures index/Eng. Prop.	Index	Index	Engineering

2.2.6.1 Standard penetration test

The main purpose of the standard penetration test is to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The great merit of the test and the main reason for its widespread use is that it is simple and inexpensive. The soil strength parameters which can be inferred are approximate, but may give a useful guide in ground conditions where it may not be possible to obtain borehole samples of adequate quality like gravels, sands, silts, clay containing sand or gravel and weak rock. In conditions where the quality of the undisturbed sample is suspect, very silty or very sandy clays, or hard clays, it is often advantageous to alternate the sampling with standard penetration tests to check the strength. If the samples are found to be unacceptably disturbed, it may be necessary to use a different method for measuring strength like the plate test [43].

When the test is carried out in granular soils below groundwater level, the soil may become loosened. In certain circumstances, it can be useful to continue driving the sampler beyond the distance specified, adding further drilling rods as necessary. The usefulness of SPT results depends on the soil type, with fine grained sands giving the most useful results, with coarser sands and silty sands giving reasonably useful results, and clays and gravelly soils yielding results which may be very poorly representative of the true soil conditions. This condition will often increase the standard penetration value. The SPT is used to provide results for empirical determination of a sand layer's susceptibility to earthquake liquefaction [43].

Standard penetration tests (SPT) are performed following TS 5744 (ASTM D-1586). The SPT is a dynamic soil penetration test that provides information on soil density and consistency. Penetration into the soil layer is commonly provided by a 140 lb hammer falling a distance of 30 inches. To quantify penetration, the number of blows needed to penetrate the soil layer every six inches for a total distance of 18 inches is counted. The number of blows counted for the last 12 inches is reported as the blow count or N value. The SPT rods are advanced by drilling to a desired test depth. The drilling produced cuttings and unwanted soil disturbance when approaching the test depth and this is reason why the first six inch blow counts is not used in the N value. As the SPT advances from the hammer blows soil is forced into a split barrel or split spoon sampler that is attached to the SPT rods.

To determine the hammer energy or efficiency, which is the ratio of the hammer's kinetic to potential energy, a section of SPT rod is instrumented with a pair of accelerometers and strain gages and then added to the rod string. Shown in Figure 2.11 is the automatic trip hammer and instrumented SPT rod. Figure 2.12 shows a typical energy efficiency plot [43].

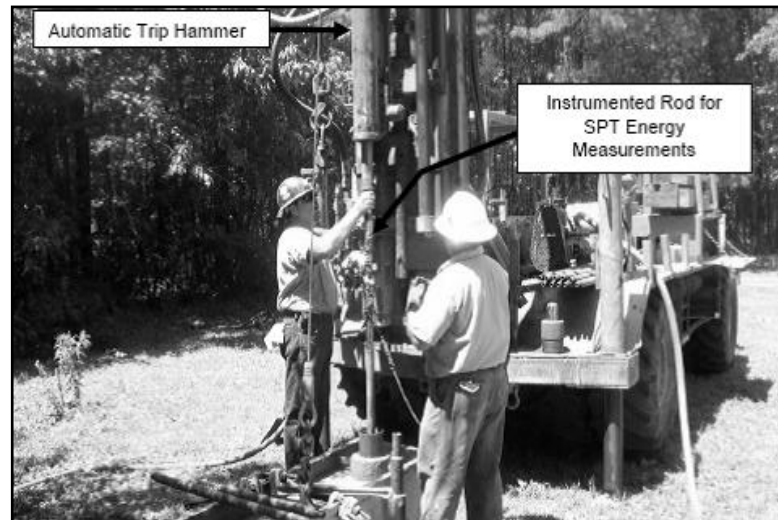


Figure 2.11 : Photograph of setup for SPT with energy measurements at site [43].

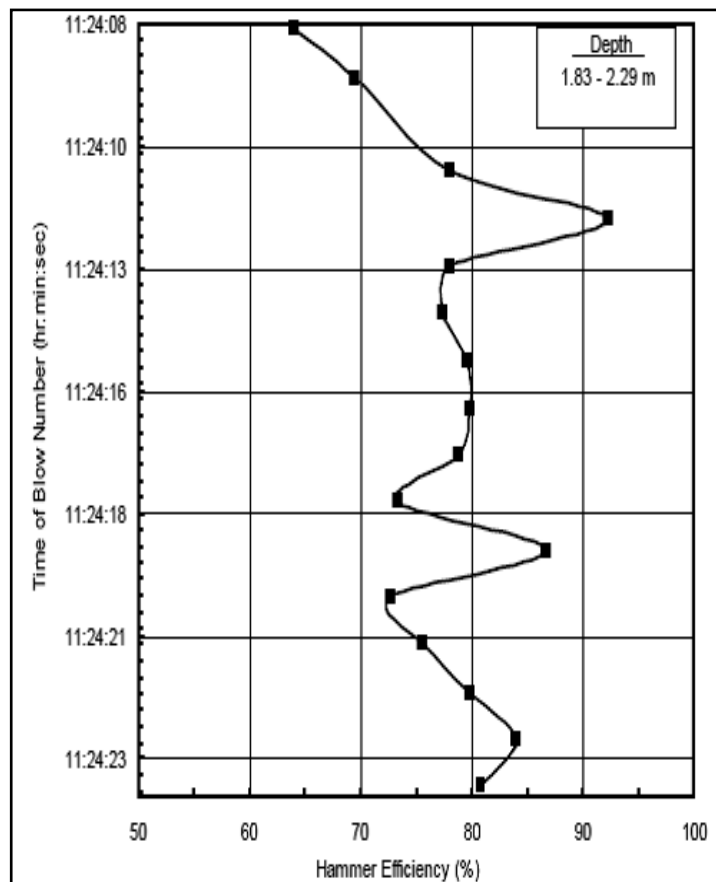


Figure 2.12 : Typical energy measurement plot for SPT boring [43].

The SPT-N values are corrected for hammer efficiency, borehole diameter, and a sampler correction. A rod length correction was not needed because the effect of rod length is included in the energy measurement. Short rod lengths may cause a reduction in hammer energy. This correction yields a N value corrected for field conditions (N_{60}) that is given by the following equation (2.11),

$$N_{60} = N \cdot C_N \quad (2.11)$$

The values of C_S for all tests were 1.0 because a standard split-spoon sampler was used. The N_{60} value is multiplied by an overburden stress correction factor (C_N) to obtain the stress-corrected N value ($N_{1,60}$) that is given by following equation (2.12).

$$N_{1,60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \quad (2.12)$$

where C_E is the hammer efficiency, C_B is the borehole diameter correction and C_S is the sampler correction [43].

Another typical setup of SPT is given in Figure 2.13. N value, which gives an indication of the soil stiffness, can be empirically related to such engineering properties as relative density, shear strength and liquefaction potential. The N value measured in the field actually reflects the influence of the soil properties, the effective confining pressure and the actual hammer energy. It has to eliminate the influence of the latter two factors by using normalized penetration resistance $N_{1,60}$ before it can be used as index of liquefaction potential [43].

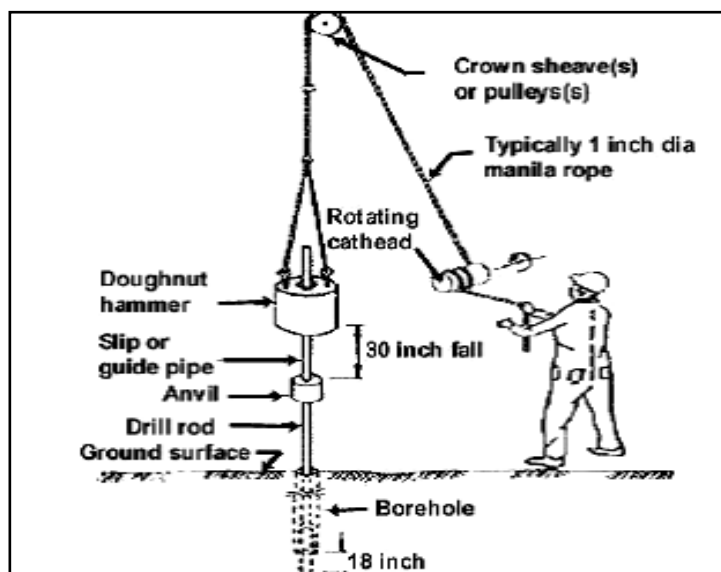


Figure 2.13 : Typical setup of SPT [43].

By using normalized penetration resistance $N_{1,60}$ before it can be used as index of liquefaction potential (2.13).

$$(N_1)_{60} = N_m C_N \frac{E_m}{0.6 E_f} \quad (2.13)$$

where N_m = the measured penetration resistance in the field, C_N = correction factor to a common reference effective overburden stress, E_m = actual hammer energy, E_f = the theoretical free-fall hammer energy (an energy ratio of 60%) [9].

An empirical formula to calculate C_N is the following equation (2.14).

$$C_N = \sqrt{\frac{1}{\sigma'_{v0}}} \quad (2.14)$$

where σ'_{v0} is the effective overburden pressure in ton per square foot. The maximum value of σ'_{v0} is suggested to be 1.7 [9].

The correlation between cyclic stress ratio causing liquefaction in the field and normalized penetration blow count was suggested by Seed from field data of Niigata earthquake 1964, Haicheng 1974 and Tangshan 1976 earthquakes, Guatemala 1976, Argentina 1977 and Miyagiken-oki 1978 earthquakes. All these field data were plotted together to provide a data base from which a boundary line separating liquefied sites from non-liquefied sites in a series of earthquakes with magnitude of 7.5 was determined [44].

The original curve was developed to estimate the liquefaction potential of pure sands. Seed studied 125 sites in North and South America, Japan and China and presented all data $N_{1,60}$ as vs. cyclic stress ratio [44].

Curves were developed for granular soils with the fines contents of 5% or less, 15% and 35%. Scaling factors were used to adjust field data from earthquakes with magnitude other than 7.5 to correspond to a magnitude of 7.5. Several changes to the SPT criteria are suggested by Youd. The trajectory of the clean sand base curve at low $N_{1,60}$ is projected to an intercept of about 0.05. Figure 2.14 is the latest chart of SPT criteria for liquefaction potential [45].

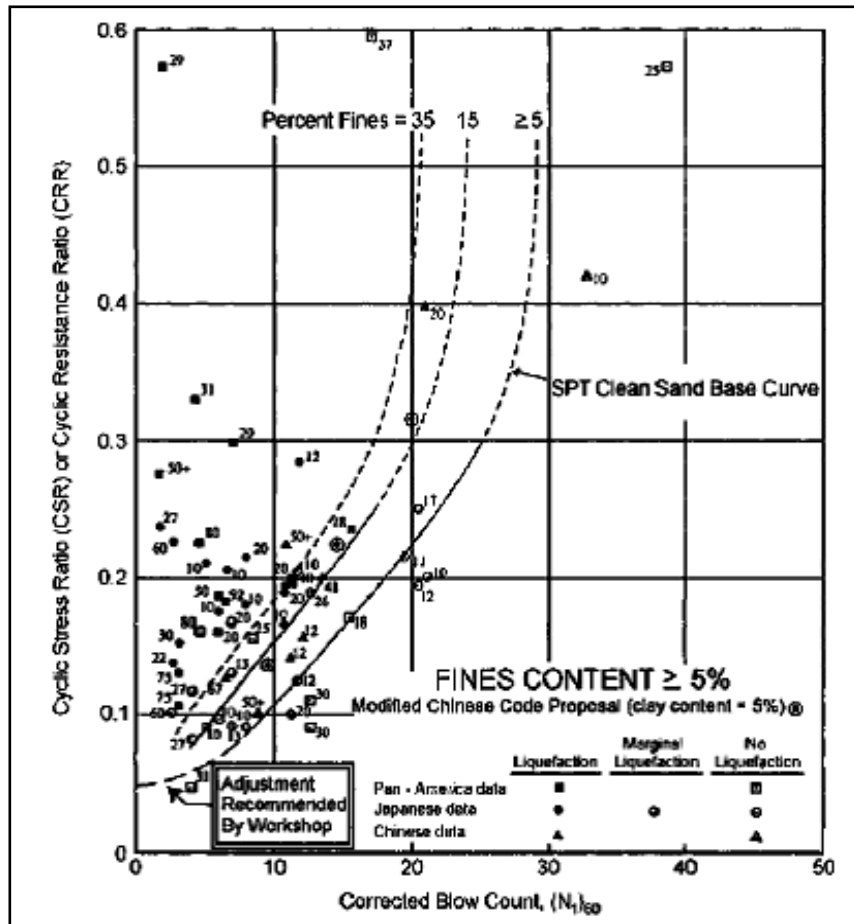


Figure 2.14 : Relationship between normalized SPT count and CRR [45].

The correction for “short” rod length between the driving hammer and the penetrating sampler was taken as a nonlinear “curve” (Figure 2.15).

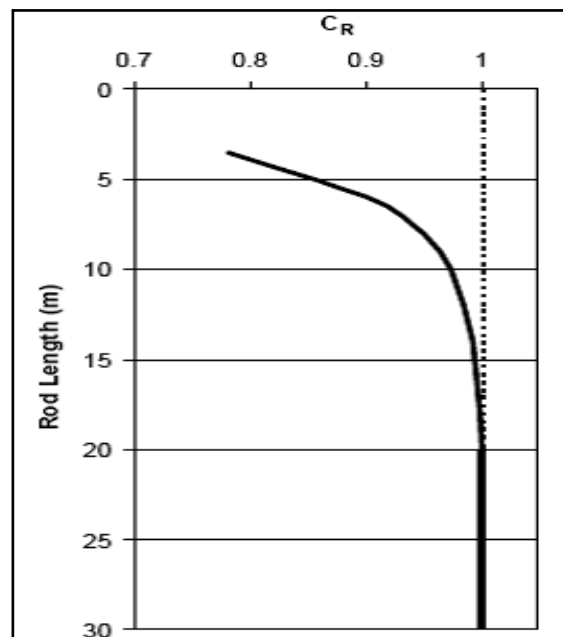


Figure 2.15 : Recommended C_R values [45].

There are several correction types for this test. Some of the corrections are given below in Table 2.6.

Table 2.6 : Recommended corrections for SPT equipment energy procedures [46].

Factor	Term	Equipment Variable	Correction
Overburden Pressure	C_N	-	$(P_a/\sigma'_v)^{0.5}$
			$C_N \leq 2$
Energy Ratio	C_E	Safety Hammer	0.60-1.17
		Donut Hammer	0.45-1.00
Borehole Diameter	C_B	65-115 mm	1.00
		150 mm	1.05
		200 mm	1.15
Rod Length	C_R	3-4 m	0.75
		4-6 m	0.85
		6-10 m	0.95
		10-30 m	1.0
		>30 m	<1.0
Sampling Method	C_S	Standard Sampler	1.0
		Sampler without liners	1.15-1.30

2.2.6.2 Cone penetration test

Cone penetration tests (CPT) has become a popular tool for rapid exploration of shallow subsurface conditions, enabling the real time collection of geotechnical and geoenvironmental data. It employs various sensors, embedded into a mechanical cone, that collects data related to soil in-situ properties and the pore fluid, as the probe is advanced into the subsurface. The early applications of CPT mainly determined the soil geotechnical property of bearing capacity. The original cone penetrometers involved simple mechanical measurements of the total penetration resistance to pushing a tool with a conical tip into the soil. Different methods were employed to separate the total measured resistance into components generated by the conical tip (the "tip friction") and friction generated by the rod string. A friction sleeve was added to quantify this component of the friction and aid in determining soil cohesive strength in the 1960s. Electronic measurements began in 1948 and improved further in the early 1970s. Most modern electronic CPT cones now also employ a pressure transducer with a filter to gather pore water pressure data. The filter is usually located on the cone tip (called U_1 position), immediately behind the cone tip (the most common U_2 position) or behind the friction sleeve (U_3 position). Pore water pressure data aids determining stratigraphy and is primarily used to correct tip friction values for those effects.

CPT testing which also gathers this piezometer data is called CPTU testing. CPT and CPTU testing equipment generally advances the cone using hydraulic rams mounted on either a heavily ballasted vehicle or using screwed-in anchors as a counter force. Typically, CPT consists of pushing a standard cone (Figure 2.16) in to the ground at a rate of 10 to 20 mm/sec. and recording the resistance during the probe advance. The probe is instrumented with strain gauges to measure the loads. The data usually recorded are the cone tip resistance (q_c), and the side friction resistance (f_s) and a continuous profile of these two parameters is plotted against depth. A computer is typically used to monitor the advancement of the probe for data acquisition, and further analysis [47].

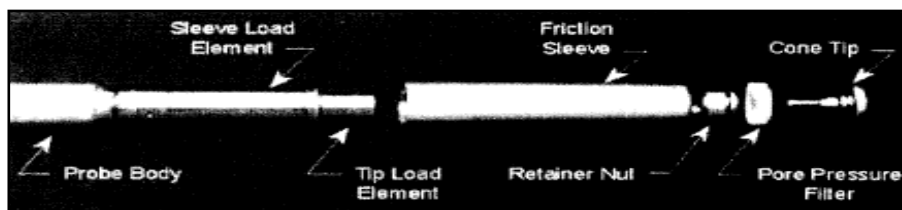


Figure 2.16 : Standard cone used for cone penetration test [49].

Further advancement in the technology enabled the measurement of pore water pressure and determination of shear wave velocity. The specialized cone penetration tests used for these purposes are called the piezocone penetration test (PCPT) and the seismic cone penetration test (SCPT). The continuous test profiles obtained with CPT allow the user to clearly explore the stratigraphy, to evaluate soil type, to estimate a large number of fundamental soil parameters, and to directly design shallow and deep foundations subjected to external loads. Common geotechnical properties which can be evaluated from cone data include horizontal stress, relative density, friction angle, constrained and Young's modulus, stress history, undrained shear strength, coefficient of consolidation, permeability, and liquefaction potential. In recent years, the CPT has been widely used in the investigation of the in-situ characteristics of soil deposits. A primary advantage of CPT is that a continuous profile of penetration resistance can detect the presence of thin layers. The CPT results are more consistent and repeatable than results from SPT. The CPT based criteria for clean sand can be illustrated graphically, as shown in Figure 2.17, from CPT liquefaction case history data. The chart shows the relationship of normalized CPT tip resistance and the cyclic resistance ratio. The CRR curve conservatively separates liquefied region from non-liquefied region [47].

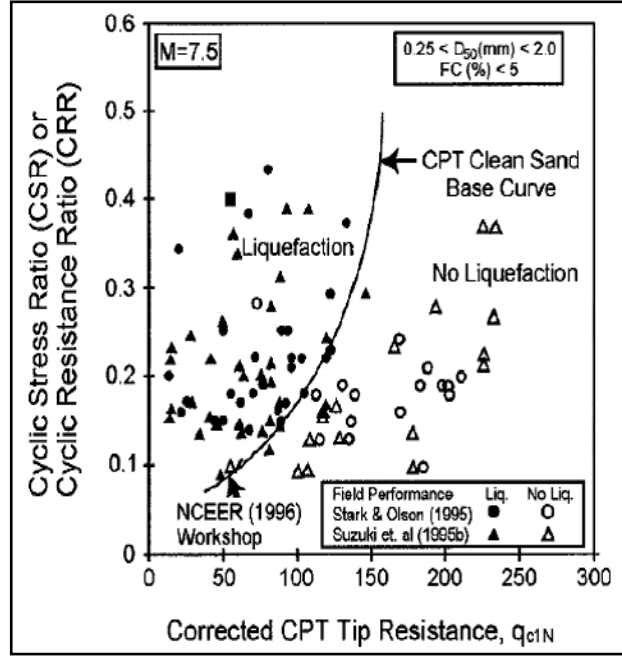


Figure 2.17 : Relationship between normalized CPT tip resistance and CRR [47].

Although the benefits from CPT lie primarily in its ability to detect thin layers of loose soil, the database of sites of observed liquefaction remains fairly small when compared with SPT results. In CPT based evaluation of liquefaction potential, the tip resistance (q_c) is usually normalized to P_a (1 atm) to obtain a dimensionless quantity (2.15) [36, 47]

$$q_{cN} = \frac{q_c}{P_a} \quad (2.15)$$

After adjusting q_c as a dimensionless value q_{cN} , further management is usually applied to normalize q_{cN} to standard effective overburden pressure of 1 ton/ft² (2.16).

$$q_{c1N} = C_N \cdot q_{cN} \quad (2.16)$$

where C_N is a normalizing factor, which has different expressions and notations in literature. The purpose of the overburden normalization is to obtain quantities that are more uniquely relate to the sand's relative density (D_R) [36, 47].

The CPT offers advantages with regard to cost and efficiency (as no borehole is required). A second advantage is consistency, as variability between equipment and operators is small in contrast to SPT. The most important advantage of CPT, however, is continuity of data over depth [47].

SPT can only be performed in 18 inch (46cm) increments, it is necessary to advance and clean out the borehole between tests. Accordingly, SPT can only be performed at vertical spacing's of about 30 inches (75cm) or more. As a result, SPT can completely miss thin liquefiable strata between test depths. Similarly, with a 12 inch test height and allowance for effects of softer overlying and underlying strata, SPT can fail to suitably characterize strata less than about 3 to 4 feet in thickness. CPT, in contrast, is fully continuous and nothing. The need to penetrate about 4 to 5 diameters into a stratum to develop full tip resistance, to be at least 4 to 5 diameters from an underlying softer stratum, and the “drag length” of the following sleeve, cause the CPT test to poorly characterize strata of less than about 12 to 15 inches (30 to 40cm) in thickness, but this allows for good characterization of much thinner strata than SPT. For a thin layer to be adequately characterized, the CPT provides indications of potentially problematic soils if one examines the q_c and f_s traces [47].

Owing to its attractive form and simplicity, the CPT based correlation of Robertson and Wride are increasingly used for liquefaction studies.

A composite parameter IC is a measure of the distance (the radius) from a point above and to the left of the plot of normalized tip resistance ($q_{c,1}$) and normalized Friction Ratio (F) as indicated in Figure 2.18. Tip resistance is corrected for increasing fines content and plasticity as given in (2.17).

$$q_{c,1mod} = q_{c,1} \times K_C \quad (2.17)$$

The recommended “fines” correction is a nonlinear function of IC, and ranges from a multiplicative factor of $K_C = 1.0$ at $IC = 1.64$, to a maximum value of $K_C = 3.5$ at $IC = 2.60$. Based on cross comparison with the new SPT-based correlation, it appears that the CPT based correlation of Robertson and Wride is slightly unconservative for clean sands, especially at high CSR, and that it is highly unconservative for soils of increasing fines content and plasticity [47].

In the figure zone A is representing cyclic liquefaction possible - depends on size and duration of cyclic loading, in the zone B for liquefaction unlikely – check other criteria, and in the zone C for flow / cyclic liquefaction possible – depends on soil plasticity and as well as size and duration of cyclic loading.

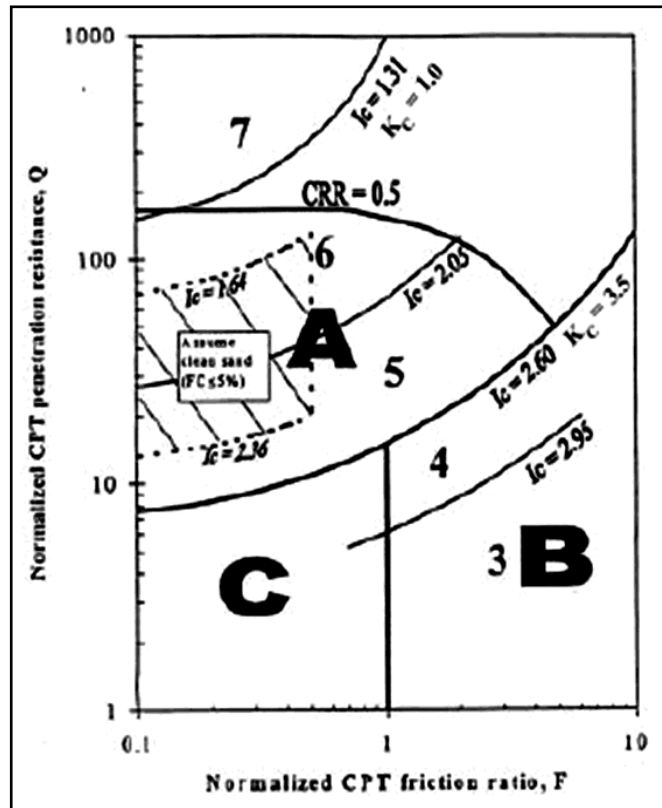


Figure 2.18 : Fines correction by CPT friction ratio to CPT penetration resistance graph [47].

2.2.6.3 Shear wave velocity measurement tests

The shear wave velocity (V_s) in soils provides a useful mean of estimating various soil properties such as void ratio, effective confining pressure, stress history and geologic age. Both V_s and liquefaction resistance have been shown to depend on the above mentioned properties. This justifies the usefulness of using V_s , as an index for evaluating the liquefaction resistance of soil. Some of the advantages of using shear wave velocity for evaluation of liquefaction potential are as follows; the nondestructive and nonintrusive techniques can be used in a region where borings and soundings are not permitted and V_s is directly related to the small strain shear modulus G_{max} , a parameter required for the estimation of dynamic shearing strain in soil [48].

However, the use of V_s for evaluation of liquefaction potential is faced with the following limitations; the field seismic measurements are made with small strains, whereas liquefaction is a large strain phenomenon and no soil samples are obtained during the tests, for classification of soil and identification of non liquefiable, soft clay rich soils [48].

Nonetheless the methodologies developed based on Vs data have been shown to be very useful for the prediction of liquefaction potential. A number of in-situ tests such as cross hole seismic test (CHST), down hole seismic test (DHST), the seismic cone penetration test (SCPT), spectral analysis of surface waves (SASW), PS Logging test, microtremor array method (MAM) and refraction microtremor method (REMI) are used to obtain the shear wave velocity profiles of the subsurface layered soils.

Crosshole test method determines shear and compressional velocities of undisturbed soils and rock strata for calculation of dynamic properties. Dynamic soil property values are used in liquefaction studies, earthquake engineering, and for assessing seismic stability of soils for construction purposes. In crosshole test, three component wall locking geophones and a directional downhole seismic source are initially lowered to the bottom of two boreholes. The two probes are then moved together in intervals of 300 – 600 mm. so that a near horizontal ray path is maintained between them. Average shear S and compressional wave P velocity values are obtained by calculating wave travel times between the source and receiver boreholes. For this test, 75 to 100 mm ID cased boreholes must be drilled to the depths of interest. The borehole must be drilled as vertical as possible, cased with steel or PVC pipes, capped at the bottom, and grouted according to the ASTM standard D4428 for the crosshole seismic testing [48].

The advantages of crosshole seismic test include fast field setup time for multiple surveys around the test structure, topographic techniques for 2 dimensional imaging of the soil between boreholes and assessment of the effectiveness of soil improvement techniques.

The seismic cone penetration test (SCPT), uses a standard cone penetrometer equipped with two triaxial set of geophones, one set located behind the friction sleeve, and the second set located exactly one meter above the first set. The configuration is also equipped with a bidirectional inclinometer. During SCPT, the cone is pushed down in one meter intervals. At each interval, a seismic shear wave source is activated at the ground surface. The seismic signal travels downwards into the ground and is recorded by both sets of geophones, and the data is acquired by a multichannel seismograph. The travel time to the upper set of geophones will be less than the travel time to the lower set due to the shorter distance the seismic signal has to travel [48].

The difference in time will be the time the signal takes to travel in the soil between the upper and the lower geophone set. Distance between the two geophone sets divided by the difference in travel time gives velocity of propagation of seismic wave in the layer between the two geophone sets. Using the successive readings at one meter intervals following the above mentioned procedure, a vertical profile of both compressional and shear waves can be obtained. The vertically oriented geophones measure compression wave velocity and the horizontally oriented geophones measure the shear wave velocities [48].

The spectral analysis of surface waves (SASW) method is a relatively new in-situ seismic method for determining shear wave velocity profiles. Testing is performed on the ground surface, which makes the method nondestructive and nonintrusive, allowing for less costly measurements than with traditional borehole methods. The basis of SASW method is the dispersive characteristic of Rayleigh waves when travelling through a layered medium. The shear wave velocity profiles are determined from the experimental dispersion curves obtained from SASW measurements through a process called forward modeling or through an investigation process [48].

Once the shear wave velocity profiles are determined, the shear and Young's module of the materials can be calculating liquefaction. There are many examples of serious damage caused by liquefaction in earthquakes such as the sudden loss of bearing capacity of foundations, dislocation of retaining walls and the excessive lateral soil movement [48]. Because of this importance, the field techniques based on the shear wave velocity are developed in the past.

The field technique for conducting SASW tests is described by Ballard and McLean and Robertson. The SASW method requires an accessible surface for receiver attachments. The extent of the accessible surface limits the investigation depth. Typically, to obtain the material properties to a depth D , the accessible surface should extend into the line of receivers direction to a distance equal to $1.5D$ and preferably $2D$; the general field arrangement used in SASW testing is depicted in Figure 2.19 [49].

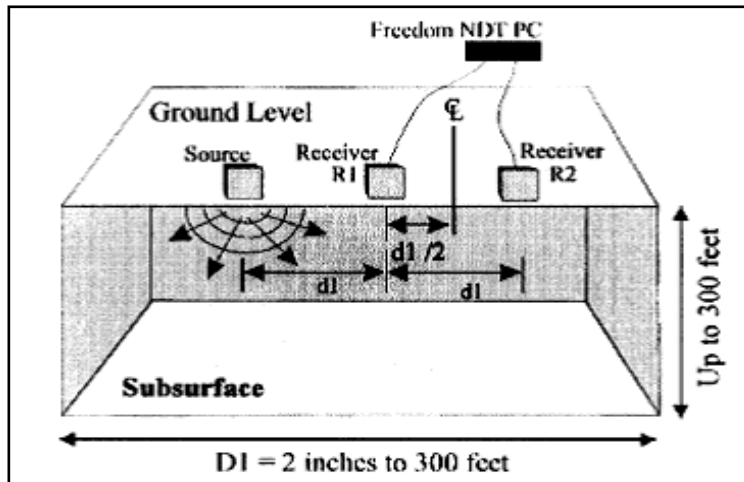


Figure 2.19 : Field setup for SASW tests [49].

In SASW tests, two receivers are placed on the surface, and a hammer is used to generate the wave energy. Other sources used in SASW measurements include solenoid-operated impactors and V meters, large drop weights and bulldozers. Short receiver spacing's are used to sample the shallow layers while long receiver spacing's are used in sampling the deep materials. A signal analyzer is used to collect and transform the receiver outputs to frequency domain. This SASW method can be used to obtain shear wave velocity profiles for earthquake site response, liquefaction analysis. In-situ shear wave velocity are very attractive because: (1) V_s can be measured with non-intrusive methods (e.g. Spectral Analysis of Surface Waves (SASW)), and (2) V_s can be measured in coarse soils (gravelly soils and coarser) in which SPT and CPT can be obstructed by interference with coarse soils particles. V_s based correlations can provide a potentially rapid screening method [49]. At this time, best V_s based correlation available is that of Andrus and Stokoe. Figure 2.20 presents core of this correlation, which is based on overburden stress-corrected $V_{s,1}$ versus magnitude-correlated equivalent uniform CSR.

Although it is certainly the best of its type, this correlation is less well defined than either SPT- or CPT-based correlations. It is also because V_s does not correlate as reliably with liquefaction resistance as does penetration resistance because V_s is a very small-strain measurement and correlates poorly with a much "larger strain" phenomenon (liquefaction). Small amounts of "ageing" and cementation of interparticle contacts can cause V_s to increase more rapidly than the corollary increase in liquefaction resistance [49].

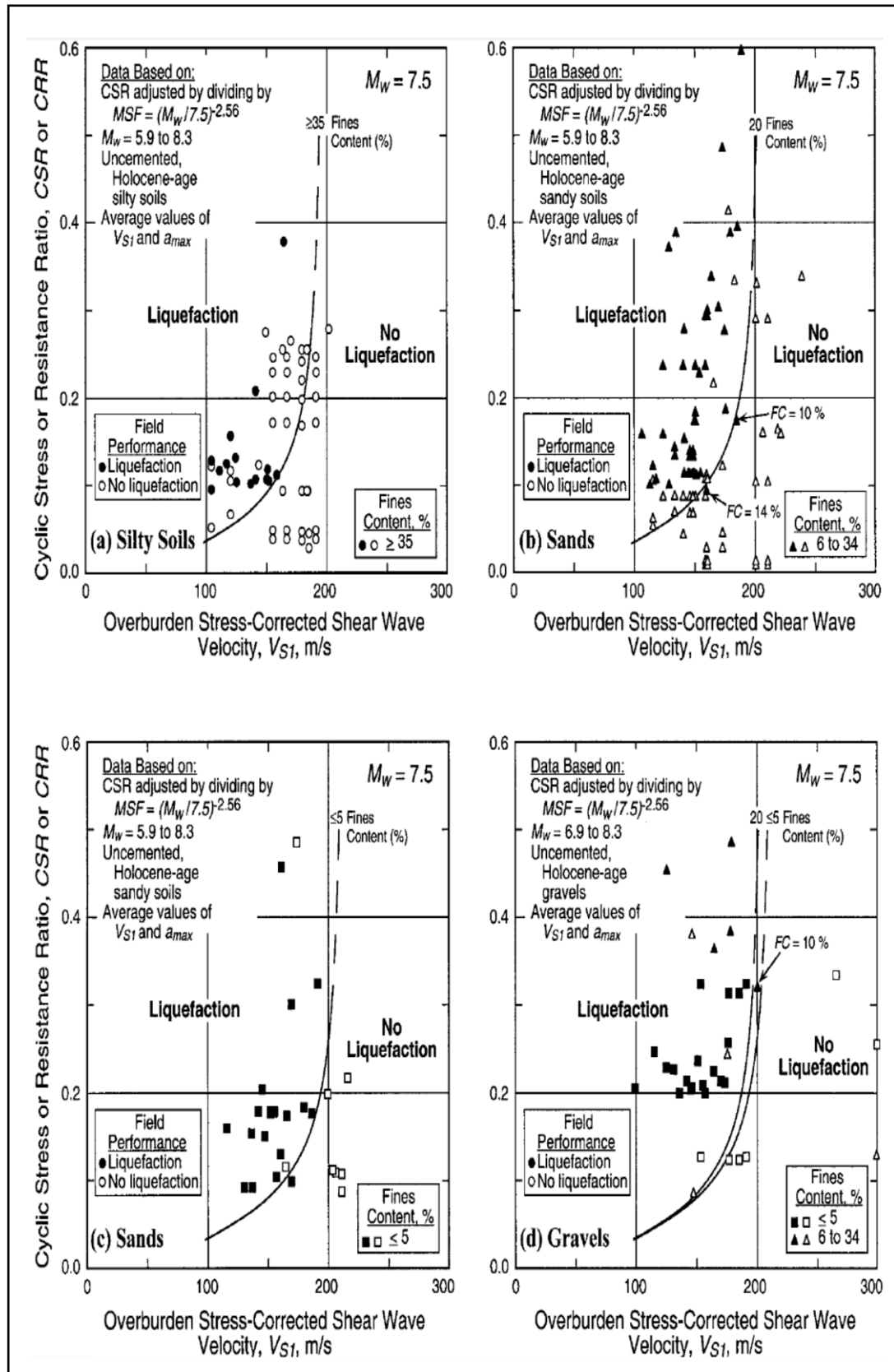


Figure 2.20 : Shear wave velocity based liquefaction triggering correlation [50].

In conclusion, a comparison of SPT, CPT, and V_s based methods of liquefaction resistance is addressed in Table 2.7.

Table 2.7 : Comparison of in-situ tests [9].

Feature	Test Type			
	SPT	CPT	V _s	BPT
Data base from past EQ's	Abundant	Abundant	Limited	Sparse
Type of stress-deformation in test	Partly, drained, large strain	Drained, large strain	Small Strain	Partly drained, large strain
Quality control, repeatability	Poor to good	Very good	Good	Poor
Detection of heterogeneity	Good if tests closely spaced	Very good	Fair	Fair
Most suitable soil types	Gravel free	Gravel free	All	Gravelly soil
Soil sample obtained	Yes	No	No	Possibly
Index value or property measured directly	Index	Index	Property	Index
Data suitable for theoretical interpretation/analysis	No	Yes	Yes	No

2.2.7 The return period of soil liquefaction

The methods of liquefaction analysis most commonly used by geotechnical engineers, including the methods, do not provide information necessary for complete, accurate, and consistent hazard analysis because they consider only a single level of ground shaking and their result is conditional upon that level of shaking. For example, a conventional liquefaction assessment might use 475 year return period loading but this does not provide an estimate of the total 475 year liquefaction hazard because total liquefaction hazard at that return period is composed of contributions from all return periods of ground shaking stronger motions that occur less frequently and weaker motions that occur more frequently [52].

For this purpose, performance based liquefaction analysis utilize the entire ground motion hazard curve, thus, incorporate the contributions from all return periods of ground shaking into an estimate of the return period of soil liquefaction. Estimating the return period of liquefaction by using performance based analysis, it requires two key sources of information. One is a hazard curve describing the mean annual rate of exceedance of the full range of ground motion intensities at the site, and the other is a set of fragility curves describing the probability of liquefaction for a given level of ground motion intensity [52].

The first requirement is satisfied by the results of a probabilistic seismic hazard analysis (PSHA) for the site using the intensity measure of interest, and the second requirement is satisfied by conventional probabilistic liquefaction analysis.

After, these requirements solved, we can estimate the return period of liquefaction by using performance based analysis due to two key sources that are solved before [52]. In conclusion, we have to consider the return period for liquefaction analysis and have to estimate the return period of liquefaction maps for these areas.

2.2.8 Mitigation procedures of liquefaction

Many catastrophic events during the past earthquakes such as loss of human life and severe damage to a large number of structures are related to the soil failure and occurrence of large displacements caused by soil liquefaction. Therefore, in order to reduce the detrimental consequences of this phenomenon, liquefaction countermeasures should be taken into account. In general, it is not recommended to build a structure or a facility in a region highly vulnerable to liquefaction; however, sometimes due to limitations in site selection, structures are built in sites with potentially liquefiable soils. Regardless of avoiding potentially liquefiable sites, the following measures can be applied to reduce liquefaction hazards. These are considering liquefaction resistant structural elements in the design and construction of structures, using deep foundations, designing containment or encapsulation structures, replacing the liquefiable soil deposits with more resistant materials, providing drainage systems and improving the soil strength characteristics [53].

The mitigation measures and soil treatment engineering is a vast science including a combination of theory and experience, and it is not possible to discuss it in detail here. As a summary, the concept of mitigation methods against liquefaction can be directly understood from the key phrase in definition of the phenomenon. Because of liquefaction has a relationship with pore water pressure, liquefaction hazards can be reduced either by promoting drainage of the soil deposits to prevent excess pore water pressure buildup or by diminishing the tendency of the soil deposit to contract by means of soil densification [53].

Remediation objectives include increasing soil's liquefaction resistance through densification, increasing its strength and improving its drainage [54]. Use of these measures has limited occurrence of liquefaction during recent earthquakes. Table 2.8 presents most common remediation measures.

Table 2.8 : Liquefaction counter measures [54].

METHOD	PRINCIPLE	Most Suitable Soil Conditions Or Types	Maximum Effective Treatment Depth	Relative Costs
1) Vibratory Probe a) Terraprobe b) Vibrorods c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); > 30 m; vibrowing, 40 m.	Moderate
2) Vibro-compaction a) Vibrofloat Vibro-Composer system.	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	> 20 m	Low to moderate
3) Compaction Piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	> 20 m	Moderate to high
4) Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best, other types can also be improved.	30 m (possibly deeper)	Low
5) Displacement (compaction grout)	Highly viscous grout acts as radial hydraulic jack when pumped in under high press.	All soils.	Unlimited	Low to moderate
6) Surcharge or buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	Dependent on size of surcharge/buttress	Moderate if vertical drains are used
7) Drains a) Gravel b) Sand c) Wick Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand > 30 m; depth limited by vibratory equipment; wick, > 45 m	Moderate to high
8) Particulate grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout methods
9) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarse.	Unlimited	High

Table 2.8 (continued): Liquefaction counter measures [54].

10) Pressure injected lime	Penetration grouting – fill soil pores with lime.	Medium to coarse sand and gravel.	Unlimited	Low
11) Electro kinetic injection	Stabilizing chemical moved into and fills soil pores by electro-osmosis or colloids into pores by electrophoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive
12) Jet grouting	High-speed jets at depth excavate, inject and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
13) Mix-in-place piles and walls	Lime, cement or asphalt introduced through rotating auger or special in-place mixer.	Sands, silts, clays, all soft or loose inorganic soils.	> 20 m (60 m obtained in Japan)	High
14) Vibro-replacement stone and sand columns a) Grouted b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	> 30 m (limited by vibratory equipment)	Moderate
15) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear and compression.	All soils.	Unknown	Moderate to high
16) Blasting	Shock waves and vibrations cause limited liquefaction, displacement, remolding and settlement to higher density.	Saturated, clean sand; partly saturated sands and silts after flooding.	> 40 m	Low

Basically, the best way to assess the performance of a mitigation method for a particular site subjected to an earthquake with certain intensity is to compare the observed seismic induced effects in the mitigated and unmitigated zones of the site.

So for the liquefaction hazard mitigation, we have to compare the observed seismic induced effects in the mitigated and unmitigated zones. Liquefaction hazards have typically been mitigated using two types of techniques: usage of specialized foundation types and in situ soil improvement. Soil improvement techniques typically involve some type of soil densification process, such as vibro-compaction, deep soil mixing, dynamic compaction, or compaction grouting, as illustrated in Figure 2.21.

These techniques tend to compact the soil, reducing the tendency for contraction during an earthquake, and thus reduce the effects of liquefaction. Although these techniques are generally effective, they are also relatively expensive and time-consuming [55].

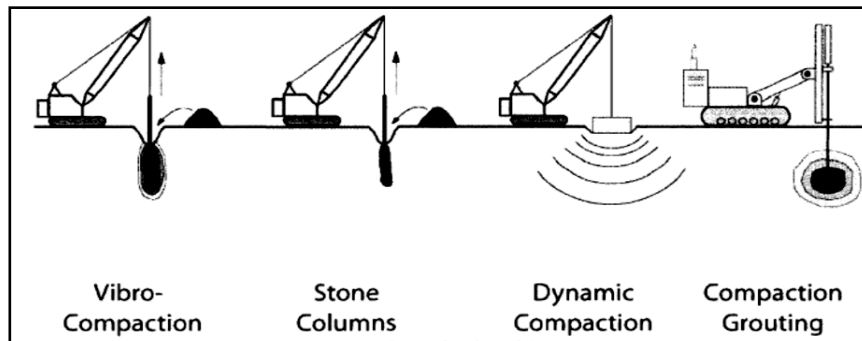


Figure 2.21 : Typical soil improvement techniques for densifying loose saturated sands to prevent earthquake induced liquefaction [56].

An alternative to soil densification is to provide for rapid pore pressure dissipation to prevent liquefaction. Seed and Booker pioneered the development of vertical gravel drains for that purpose. Vertical drains allow for pore pressure dissipation through horizontal flow, which significantly decreases the drainage path length. When drainage is impeded by a horizontal silt or clay layer, vertical drains can be particularly effective. The effectiveness of the drains increases as drain diameter increases and / or drain spacing decreases [57].

Although gravel drains have been utilized at many sites for liquefaction mitigation, most designers have relied on the densification caused by drain installation rather than the drainage which they provide. Some designers have worried that the use of drainage alone would still allow unacceptable settlement to develop. In as much as soil compressibility of a soil of given relative density increases as excess pore pressure increases (Figure 2.22), the key to limiting settlement appears to be in keeping excess pore pressures below a given threshold [58].

As shown in Figure 2.22, this threshold is located at an excess pore pressure ratio, equal to 0.40. Below this threshold, the compressibility of the soil is negligible, above this threshold, soil compressibility increases markedly and excessive settlement could result [58].

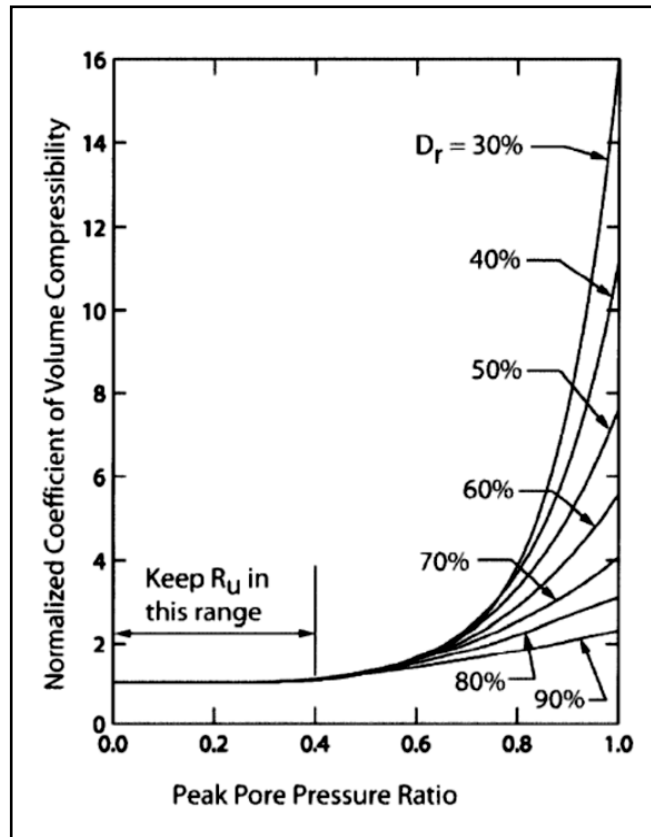


Figure 2.22 : Normalized coefficient of volume compressibility versus pore pressure ratio for sands at various relative densities where settlement is negligible when peak pore pressure ratio remains below approximately 0.40 [59].

While extensive research is ongoing on analysis of liquefaction in sands and sands with fines, recent researches has focused on use of soil remediation to reduce or eliminate risks and failures from liquefaction.

Methods for ground improvement to mitigate liquefaction potential fall into four general categories those that densify loose soils, those that expedite drainage and dissipation of excess pore water pressure, those that provide confinement and limit lateral flow of the soil, and those that physically or chemically modify the soil to increase its strength.

Table 2.9 summarizes general methods that are used in the previous literature studies from different researchers for mitigations of seismic soil liquefaction hazard. However, current soil improvement methods to safeguard against liquefaction are often very expensive and their applications are limited to new sites, because of this the method must be chosen correctly before it is going to be applied to the potentially liquefiable site [60].

Table 2.9 : General methods for mitigations of seismic soil liq. hazard [45].

General Category	Mitigation Methods	Notes
I. Excavation and/or compaction	a) Excavation and disposal liquefiable soils b) Excavation and recompaction c) Compaction (for new fill)	
II. In-situ ground densification	a) Compaction with vibratory probes (Vibroflotation, Terraprobe, etc.) b) Dynamic consolidation (Heavy damping) c) Compaction piles d) Deep densification by blasting e) Compaction grouting	- Can be coupled with installation of gravel columns - Can also provide reinforcement
III. Selected other types of ground treatment	a) Permeation grouting b) Jet grouting c) Deep mixing d) Drains -Gravel drains -Sand drains -Pre-fabricated strip drains e) Surcharge pre-loading f) Structural fills	- Many drain installation processes also provide in-situ densification.
IV. Berms, dikes, walls, and other edge containment structures/systems	a) Structures and/or earth structures built to provide edge containment and thus to prevent large lateral spreading	
V. Deep foundations	a) Piles (installed by driving or vibration) b) Piers (installed by drilling or excavation)	- Can also provide ground densification
VI. Reinforced shallow foundations	a) Grade beams b) Reinforced mat c) Well-reinforced and/or post-tensioned mat d) "Rigid" raft	

2.3 Literature Review on Liquefaction Analysis Methods

2.3.1 Overview

These method types try to develop the relationship between the in-situ test data and the site behavior of “liquefaction” or “no liquefaction”, through numerous case history datasets of in situ tests. The results from these methods have exhibited an improvement over traditional methods. Some of the recent methods are given below.

2.3.2 Artificial neural network analysis

Artificial Neural Network (ANN) is data processing paradigms that resemble the human brain in terms of both structure and functionality. These networks, though not as capable as the human brain, can still be trained to perform useful functions. Different types of ANN architectures can be obtained by varying the number of interconnected nodes, the internal mapping functions, and learning algorithm of the network. The most common and basic feed forward network performs a nonlinear transformation of input data in order to approximate the output data. The number of input and output nodes depends upon the nature of the modeling problems, the input data representation and the form of the network output required. The number of nodes in the hidden layer is related to the complexity of the system being modeled. The interconnections within the network are such that every neuron in each layer is connected to every neuron in the adjacent layers. Each interconnection has associated with it a scalar weight which is adjusted during the training phase. The hidden layer nodes typically have a transfer function associated with them [49]. In the context of the problem of evaluation of liquefaction potential the ANN analysis can be used as the training field test data for SPT, CPT, and Vs tests , and can get the solutions for the liquefy or not.

2.3.3 Probabilistic neural network analysis

The probabilistic neural networks (PNN) are based on a quite different mathematical background, compared to the multilayered feed forward neural networks. PNN are especially based on the Bayesian classifier technique commonly used in various pattern recognition problems.

Also there is a Bayesian method and it has given below with another topic. A non-parametric estimation technique known as Parzen's windows is used to construct the class dependent probability density functions for each classification category required by Baye's theory. This enables the determination of the chance a given vector pattern lies within a given category. Combining this with the relative frequency of each category, the PNN selects the most likely category for the given pattern vector [49].

The PNN model is said to effectively explore the relationship between the independent and dependent variables without any assumptions about the relationship between various variables. The training process of PNN is regarded as the determination of a network parameter as "smoothing parameter", which maximizes the classification accuracy of input vectors [49]. This PNN model has the ability to discover the intrinsic relationship between the seismic and soil parameters and liquefaction potential by using the SPT, CPT and Vs tests, etc. The Bayesian analysis is a type of architecture which uses deterministic methods to solve probability problems and it has a mapping function approach for obtaining the output values. The Bayesian mapping function relates the factor of safety (FOS) obtained from a deterministic method to the probability of liquefaction (PL) by the Bayesian interpretation. The mapping function approach presented by Juang in 2002 is described by equation (2.18) [61]

$$P_L = \frac{f_L (FOS)}{f_L (FOS) + f_{NL} (FOS)} \quad (2.18)$$

Where $f_L(FOS)$ and $f_{NL}(FOS)$ are the probability density functions of the calculated FOS for the subsets of liquefied cases and non-liquefied cases. Using the in-situ test data based case histories, the following mapping function was obtained (2.19).

$$P_L = I / [1 + (FOS/A)^B] \quad (2.19)$$

Where A and B were the regression coefficients and FOS was the respective deterministic method. For an example, based on the SPT method the values of regression constants are given in Table 2.10.

Table 2.10 : Regression constants of Bayesian mapping function [61].

Mapping Function	A	B
SPT Based	0.8	3.5

Like the SPT method, CPT and Vs models also can be used as the mapping function. This method is useful for converting SPT, CPT and Vs based deterministic factor of safety estimates into corresponding to probability of liquefaction. For example of SPT based probability curves using Bayesian mapping function is given in Figure 2.23.

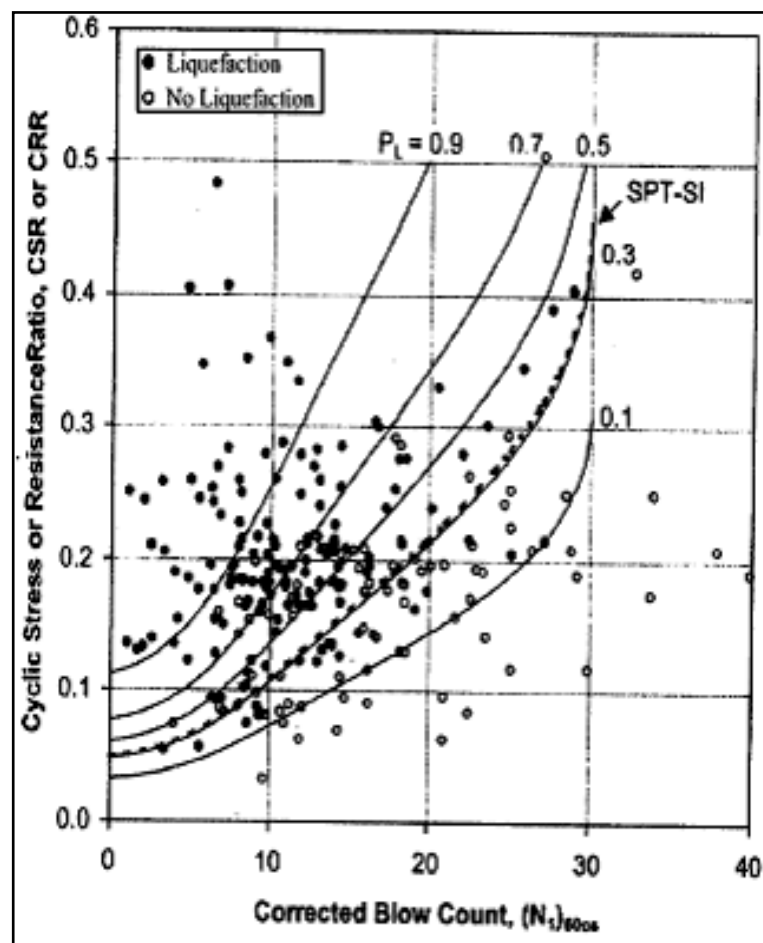


Figure 2.23 : SPT based probability curves using Bayesian map.func. [61]

2.3.4 Logistic regression analysis

Logistic Regression is a statistical tool used to examine the relationship between variables, which allows an outcome to be predicted by any number of explanatory variables. It is especially useful for continuous function approximations. It can have multidimensional input, and it will fit multidimensional surfaces through data.

When a new pattern is presented to the network that input pattern is compared in N dimensional space to all of the patterns in the training set to determine how far in distance it is from those patterns. The output that is predicted by the network, is a proportional amount of all of the outputs in the training set. The proportion is based upon how far the new pattern is, from the given patterns in the training set. The output parameter is different from the methods that are described before. The logistic regression is a type of regression that uses binary (yes / no) variables as the outcome measure. So because of this difference, the hidden layers which are carrying information between input and output layers and their relationships have a different weight [49].

In the context of evaluation of liquefaction potential, logistic regression can be used to develop mathematical equations relating the probability of liquefaction to in-situ test data such as SPT, CPT or Vs.

2.3.5 Fuzzy neural network analysis

Fuzzy neural networks (FNN) are an integrated system of fuzzy logic and artificial neural networks. It is aimed at fuzzifying some of the neural network parameters while retaining the basic properties and architectures of the neural network. The FNN model thus has advantages like learning and optimization abilities of neural network and ability to incorporate expert knowledge in the form of human like if then rules. A two stage training methodology is adopted to develop the FNN models. In the first stage, the training case histories are used to obtain initial network parameters. In the second stage of training, the training case histories are processed one by one to develop membership functions for network parameters [62].

This model is useful for situations where input variables are known only in linguistic terms and it can provide a means for preliminary evaluation of liquefaction potential for such sites where input parameters are not well defined.

2.3.6 Adaptive resonance theory based network analysis

The adaptive resonance theory (ART) based neural networks use a combination of feedback, higher level control, and nonlinearities to form clusters in state space that correspond to concepts, based on the statistical structure of the inputs.

It works over a pattern matching process that compares an external input with the pattern stored in the internal memory of the network. This process has been shown to be far more efficient and reliable than error based learning. Its networks have shown very good performance in applications involving clustering, classification, and pattern recognition. When compared to statistical and other neural network based clustering / classification algorithms, these networks usually obtain very good classification accuracy, while securing proven stability and high level of compression in the system [49].

The ART networks can be divided into two classes, the supervised learning networks and the unsupervised learning networks. The ART based neural network architectures proposed in the past are given in the Table 2.11.

Table 2.11 : ART based neural network architectures proposed [49].

S.No.	Architecture	Description	Source
1	ART1	Unsupervised learning clustering architecture for binary-valued input patterns	Carpenter and Grossberg (1987)
2	ARTMAP	Supervised learning classification architecture, for binary-valued input pairs of patterns	Carpenter et al. (1991a)
3	Fuzzy-ART	Unsupervised learning clustering architecture for analog-valued input patterns	Carpenter et al. (1991b)
4	Fuzzy-ARTMAP (FAM)*	Supervised learning classification architecture, for Analog-valued input pairs of patterns	Carpenter et al. (1992)
5	PROBART*	Supervised learning classification architecture, uses the stored, probability information for mappings involving noisy data, used for analog-valued input patterns	Marriot and Harrison (1995)
6	FAMR	FAM with a relevance factor, a generalization of PROBART	Andonie, Sasu, and Beiu (2003)
7	μ -ARTMAP	Supervised Architecture, uses a probabilistic setting, seeks, partitioning the input space, that optimizes the mutual information with the output space	Sanchez, Dimitriadis, Izquierdo, and Coronado (2000)
8	Gaussian ARTMAP	Supervised learning architecture, defining the choice and match functions to be the discriminant functions of a Gaussian classifier	Williamson, (1996)

Example architecture for supervised learning classification architecture, Fuzzy ART Map is given in Figure 2.24. This architecture learns in a supervised way, associations of pairs of analog input and output pattern and has two modes, the training mode and the testing mode. In the training phase, the network is fed with a number of input patterns and the corresponding target classes, to which these input vectors are related. The testing mode is similar to the other neural network architectures.

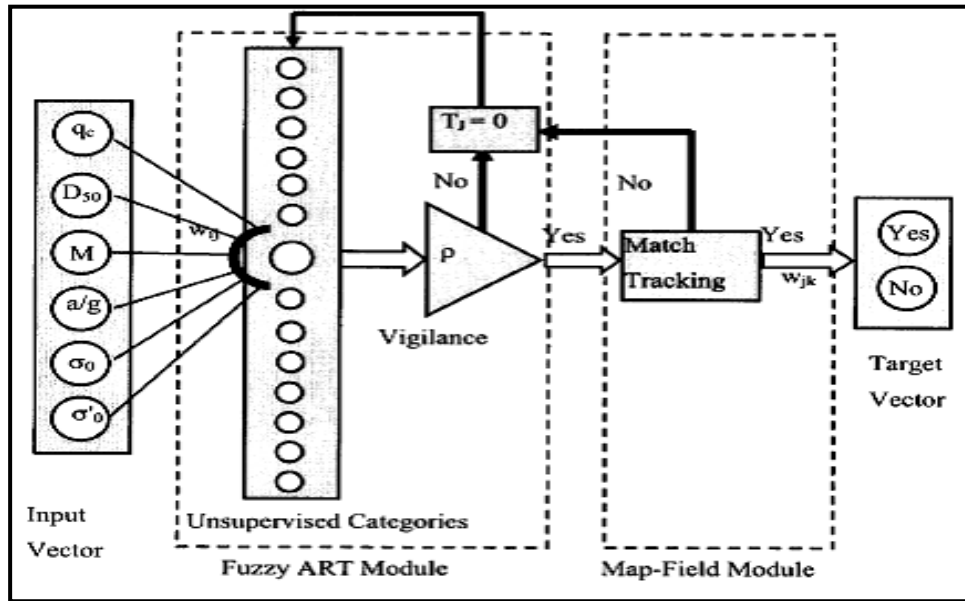


Figure 2.24 : Fuzzy ARTMAP architecture [49].

2.3.7 Instance based learning model analysis

Instance based learning (IBL) is a machine learning method used for classification and prediction. During the learning phase, IBL stores all or some of the training instance. In the testing phase, value of a new instance is required to be predicted. This is accomplished by retrieving one or more stored instances that are most similar to the new instance, which is then used to predict the value of new instance. The IBL models can be used to predict class categories and also numeric functions. IBL is a model free approach and can be used to approximate a wide range of numeric functions. In the context of liquefaction potential, the concern is prediction of classes. Various types of IBL algorithms can be adopted to build query specific local models, which attempt to fit the training examples only in a region around the query point. These algorithms can be used to understand as different bases of choosing the training examples at a closest distance to the query point presented to the model [49].

So the IBL models were said to be suitable only for well distributed data and the models were recommended to be tested before being put into liquefaction potential evaluation problem.

2.3.8 Evolutionary polynomial regression

The evolutionary polynomial regression (EPR) models for evaluation of liquefaction potential and lateral spreading are trained and tested separately using two different datasets of field case histories. It is shown that the EPR models are able to learn, with a very high accuracy, the complex relationship between liquefaction and its contributing factors in the form of a function; and generalize the learning to provide predictions for new cases that are not used in the construction of the model. Evolutionary polynomial regression (EPR) is a data driven method based on evolutionary computing, aimed to search for polynomial structures representing a system. A general EPR expression (2.20) can be presented as [63].

$$y = \sum_{j=1}^m F(Xf(X), a_j) + a_0 \quad (2.20)$$

where y is the estimated vector of output of the process; a_j is a constant; F is a function constructed by the process; X is the matrix of input variables; f is a function defined by the user; and n is the number of terms of the target expression [64].

The general functional structure represented by $f(X, a_j)$ is constructed from elementary functions by EPR using a genetic algorithm (GA) strategy. The GA is employed to select the useful input vectors from X to be combined. The building blocks (elements) of the structure are defined by the user based on understanding of the physical process. While the selection of feasible structures to be combined is done through an evolutionary process; the parameters a_j are estimated by the least's square method. EPR is a technique for data driven modeling. In this technique, the combination of the genetic algorithm to find feasible structures and the least square method to find the appropriate constants for those structures implies some advantages. In particular, the GA allows a global exploration of the error surface relevant to specifically defined objective functions. Using such objective (cost) functions some criteria can be selected to be satisfied through searching process [64].

These criteria can be set in order to (i) avoid the over fitting of models, (ii) push the models towards simpler structures and (iii) avoid unnecessary terms representative of the noise in data. EPR shows robustness and can get a model truly representative of data. An interesting feature of EPR is in the possibility of getting more than one model for a complex phenomenon. A further feature of EPR is the high level of interactivity between the user and the methodology. The user physical insight can be used to make hypotheses on the elements of target function and on its structure [64].

Liquefaction occurrence depends on the mechanical characteristics of the soil layers in the site, the depth of the water table, the intensity and duration of the ground shaking, the distance from the source of the earthquake and the seismic attenuation properties of in-situ soil. Because of the participation of a large number of factors that affect the occurrence of liquefaction during earthquake, the determination of liquefaction potential is a complex geotechnical engineering problem and has attracted considerable attention of geotechnical researchers [64].

The most influential parameters affecting the occurrence of liquefaction have been used to develop the EPR model. The same parameters have been used by some other researchers for development of traditional and pattern recognition based models [9]. These parameters include q_{c1N} , and $CSR_{7.5}$, as given in (2.21) where;

$$q_{c1N} = \frac{q_c / 100}{(\sigma'_v / 100)^{0.5}} \quad (2.21)$$

q_c = measured cone tip resistance [kPa], σ'_v = effective overburden pressure at the point of CPT measurement [kPa], CSR is defined as (2.22)

$$CSR_{7.5} = 0.65 \left(\frac{\sigma_v}{\sigma'_v} \right) \left(\frac{a_{max}}{g} \right) (r_d) / MSF \quad (2.22)$$

where σ_v = total vertical stress of the soil at a specific depth; a_{max} = peak horizontal ground surface acceleration; g = gravity acceleration; MSF = magnitude scaling factor; and r_d = shear stress reduction factor [9].

The variable r_d is calculated as (2.23).

$$\begin{aligned} r_d &= 1.0 - 0.00765z \text{ if } z \leq 9.15 \text{ m and;} \\ r_d &= 1.174 - 0.0267z \text{ if } 9.15 \text{ m} < z \leq 23 \text{ m} \end{aligned} \quad (2.23)$$

MSF is calculated as (2.24).

$$MSF = \left(\frac{10^{2.24}}{M_w^{2.56}} \right) = \left(\frac{M_w}{7.5} \right)^{-2.56} \quad (2.24)$$

The value of MSF obtained from Eq. 2.22 represents the lower bound of the range of MSF values recommended by the NCEER [9]. Equation 2.23 and 2.24 are commonly used by geotechnical researchers and practitioners, although many other formulas have been proposed for calculating r_d and MSF [65].

The variable I_c is defined as (2.25) [66].

$$I_c = [(3.47 - \log_{10} q_{c1N})^2 + (\log_{10} F + 1.22)^2]^{0.5} \quad (2.25)$$

Where F (normalized friction ratio) is given in (2.26),

$$F = f_c / (q_c - \sigma_v) \quad (2.26)$$

In equation 2.24, f_c = sleeve friction; and both f_c and σ_v are in units of kPa. It should be noted that the soil type index I_c defined in equation 2.26 is different from the soil behavior type index defined by Robertson and Wride.

2.3.9 Genetic expression programming

Genetic expression programming (GEP) of which the predecessors are the genetic algorithms (GA) and genetic programming (GP) was initially presented by Ferreira in 2001 [67].

Its evaluation system of any knowledge is like that the biological evaluation and is a computer program encoded in linear chromosomes of fixed length. In this approach, a mathematical function defined as a chromosome with multi genes is developed using the data presented to it. Although GEP mainly carries out the symbolic regression via the most of the genetic operators of GA and GP, there are some differences between GA, GP and GEP. While any mathematical expression is adopted as symbolic strings of fixed length (chromosomes) in GA, GP represents it to be nonlinear entities of different size and shapes (parse trees). But in GEP it is encoded as simple strings of fixed length which are subsequently expressed as expression trees of different size and shape [68].

GEP algorithm randomly composes an initial chromosome which represents a mathematical function and then converts it into an expression tree (ET). The subsequent step is that there is comparison between predicted values and actual values. If desired results in accord with error criteria initially selected are obtained, the GEP process is ceased. GEP has two main components called chromosomes and the expression trees (ET). The chromosomes in which coded some information using special language about the problem may have one or more genes. Any mathematical information is coded in gene in chromosomes by way of bilingual and conclusive language called Karva Language (the language of the genes) and also is translated to the expression trees by means of the language of ET. The Karva Language has significant advantages in terms of precisely inferring the genotype. Genes have two parts known the head and the tail. The head of a gene contains main variables required to code any expression such as some functions, variables and constants [68].

GEP has four primary operators such as selection, mutation, transposition, and cross-over. The selection operator selects any chromosomes using the method called roulette wheel sampling with elitism to obtain the higher probability of producing offspring. The coding sequence in selected chromosome is changed by mutation operator. In other words, the selected chromosome is mutated.

However the structure of chromosome is not changed. Transposition operator randomly copies a part of chromosome and put it into another position. The coding of randomly selected two chromosomes is exchanged between them via cross-over. The operators given above are applied with operator rate which indicates a certain probability of a chromosome. Operator rates are determined by user before analysis. The mutation rate is generally used between 0.001 and 0.1. On the other hand it is suggested that transposition rate and cross-over rate are 0.1 and 0.4 [68].

2.3.10 Finite element analysis

In the finite element method (FEM), the solution domain of a problem is idealized by a finite number of smaller domains called finite elements. For each finite element, the governing differential equation is approximated within the element using trial functions. The total solution of the problem can be found from solving a global matrix along with imposing boundary conditions [49].

Finite element method can be applied to most kinds of boundary value problems with different geometries and boundary conditions. Application of FEM in geotechnical engineering began in 1966, when Clough and Woodward used this method for analysis of embankments. In recent years, FEM has been widely used for analysis of soil liquefaction [49]. Due to rapid advances of computer technology in the recent decades, method has been developed very quickly, and it has found its place as a robust tool for analysis in liquefaction probability evaluation. Like other numerical methods FEM is a numerical approach to provide approximate solutions for differential equations. However, because of forming the problem and boundary conditions were difficult, in the past it has not so popular. But, today with advances of computer technology, liquefaction probability evaluation problems can be solved by using this method.

2.4 Neural Network Algorithm

Artificial neural networks are systems and computational devices that are constructed to make use of some organizational principles resembling those of the human brain. Normally there are a large number of highly connected computational nodes (neurons) that are operated and configured in parallel regular architectures. Like human brain an artificial neural network has the ability to learn; recall and generalize from the data which are used to train the system.

The pattern of activation at the input units represents the problem being presented to the network; and the pattern of activation at the output processing units, represents computational results achieved by neural network. The neural network propagates the changes in weights of the connection between each linked neuron by minimizing the difference between the actual output and target output. The propagation of the changes in link weight the computation performed by neural network is strongly affected by the topology and strengths of the connections between the neurons. As an example, Figure 2.25 shows a simple mathematical model of this network.

The basic building block of neural network technology is the simulated neuron. Independent neurons are of little use, unless they are interconnected in a network of neurons. The network processes a number of inputs from the outside world in order to produce an output, the network's classifications and predictions.

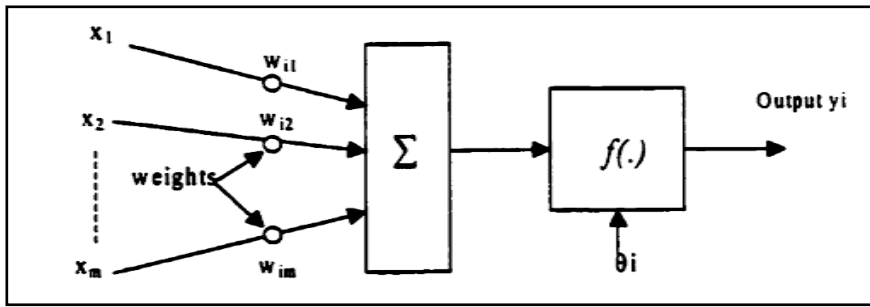


Figure 2.25 : Schematic diagram of a neuron's network [69].

The neurons are connected by weights, which are applied to values passed from one neuron on to the next. A group of neurons is called a slab. Neurons are also grouped into layers by their connection to the outside world. For example, if a neuron receives data from outside of the network, it is considered to be in the input layer. If a neuron contains the network's predictions or classifications, it is in the output layer.

Neurons in between the input and output layers are in the hidden layer(s). A layer may contain one or more slabs of neurons (Figure 2.26).

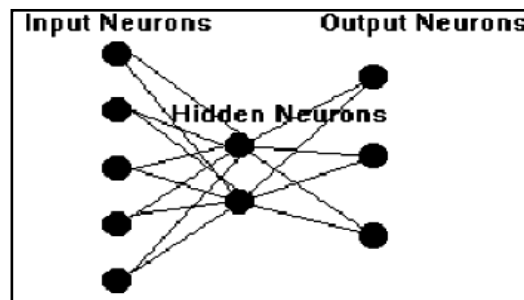


Figure 2.26 : Neural networks structure [70].

Neural networks are not programmed; they learn by example. Typically, a neural network is presented with a training set consisting of a group of examples from which the network can learn. The most common training scenarios utilize supervised learning, during which the network is presented with an input pattern together with the target output for that pattern. The target output usually constitutes the correct answer, or correct classification for the input pattern. In response to these paired examples, the neural network adjusts the values of its internal weights. If training is successful, the internal parameters are then adjusted to the point where the network can produce the correct answers in response to each input pattern. Usually the set of training examples is presented many times during training to allow the network to adjust its internal parameters gradually.

The neural network approach does not require human development of algorithms and programs that are specific to the classification problem hand, suggesting that time and human effort can be saved. There are drawbacks to the neural network approach, however: the time to train the network may not be known beforehand, and the process of designing a network that successfully solves an application problem may be involved. It is possible to develop a network that can generalize on the tasks for which it is trained, enabling the network to provide the correct answer when presented with a new input pattern that is different from the input in the training set. To develop a neural network that can generalize the training set must include a variety of examples that are good preparation for the generalization task. In addition, the training session must be limited in iterations, so that no “over learning” takes place.

Thus, special considerations in constructing the training set and the training presentations must be made to permit effective generalization behavior from a neural network. All of these characteristics of neural networks may be explained through the simple mathematical structure of neural net models. The computations performed in the neural net may be specified mathematically.

As a summary, an artificial neural network is a parallel computing system with the following characteristics of it is a naturally inspired mathematical model, it consists of a large number of highly interconnected processing units, its connections (weights) hold the information about the relationship between the inputs and outputs, each neuron can dynamically respond to its input stimulus and the response completely depends on its local information that is the input signals arrive at the processing element via connections and connection weights, it has the ability to learn recall and generalize from training data by adjusting the connection weights and also its collective behavior demonstrates the computational power while no single neuron carries specific information, the information is distributed among the neurons which have a great deal of computational power [71].

There are different types of neural network architectures. These architectures differences are their algorithm and function formulas. In detail, a typical structure of ANN's consists of a number of interconnected processing elements (PEs), commonly referred to as neurons.

The neurons are logically arranged in layers: an input layer, an output layer, and one or more hidden layers. The neurons interact with each other via weighted connections. Each neuron is connected to all the neurons in the next layer. The input layer is the means by which data are presented to the network. The output layer holds the response of the network to the input. The hidden layers enable these networks to represent and compute complicated associations between inputs and outputs [71].

The back-propagation learning algorithm is the most commonly used neural network algorithm. The back-propagation neural network has been applied with great success to model many phenomena in the field of geotechnical and geoenvironmental engineering. Each neuron in a layer receives and processes weighted inputs from neurons in the previous layer and transmits its output to neurons in the following layer through links. Each link is assigned a weight that is a numerical estimate of the connection strength. The weighted summation of inputs to a neuron is converted to an output according to a nonlinear transfer function. The common transfer function widely used in the literature is the sigmoid function. The neural network “learns” by modifying the weights of the neurons in response to the errors between the actual output values and the target output values. This is carried out through gradient descent on the sum of the squares of the errors for all the training patterns [71].

The changes in the weights are proportional to the negative value of the derivative of the error term. One pass through the set of training patterns, together with the associated updating of the weights, is called a cycle or an epoch. Training is carried out by repeatedly presenting the entire set of training patterns (updating the weights at the end of the each epoch) until the average sum-squared error over all the training patterns is minimal and within the tolerance specified for the problem.

At the end of the training phase, the neural network should correctly reproduce the target output values for the training data and provided the errors are minimal. The associated trained weights of the neurons are then stored in the neural network memory.

In the next phase, the trained neural network is feed by a separate set of data. In this testing phase, the neural network predictions using the trained weights are compared with the target output values. The performance of the overall ANN model can be assessed by several criteria.

These criteria include the coefficient of determination (R^2), mean-squared error, mean absolute error, minimum absolute error, and maximum absolute error. A well trained model should result in an R^2 value close to 1 and small values of the error terms.

2.5 Finite Difference Method

Today's another popular method is finite difference method which is useful for dealing with geotechnical dynamic problems. There are some computer programs based on finite difference method that can solve geotechnical problems, however these methods still require an interpretation of the results of analysis, and it has not widely used for general liquefaction analysis.

However, with advanced computer technology and interactive visualization of the results of such analyses, the methods have a place among the general methods used in liquefaction analysis.

Finite difference method widely used to obtain approximate solutions of many boundary value problems whose exact solutions are mathematically complex and in some cases impossible. Response of a structure system is often represented by the governing differential equations.

These equations involve derivatives of functions using finite difference approach these derivatives can be easily evaluated at discrete points. The partial differential equations (PDE) can then be solved in the domain with respect to some given boundary conditions.

Finite difference method is an approximate method for determining derivatives of a function and it may give exact results. However, frequently it yields only approximate results.

The extent of error in using finite difference method in finding derivatives of a function depends on various including order of derivative, type of function, type of finite difference mesh.

2.6 Computer Programs for Liquefaction Analysis

2.6.1 Computational tools for liquefaction analysis

LASS-II (Liquefaction Analysis of Saturated Soil Deposits) analyzes seismic response and liquefaction of horizontally layered saturated solids. The saturated soil below the water table is modeled as a coupled two - phase medium with solid granular skeleton and pore water as the constituent materials. Coupling between these two phases is taken into account.

The pore water is allowed to flow with respect to granular solid and this process is assumed to be governed by Darcy flow law with the coefficient of permeability as the material constant. Above the water table, soil is modeled as one phase solid. A nonlinear material model is used in the program which includes yielding, failure, volume change characteristics, cyclic effects and criteria for initial and final liquefaction. Two different material models are used for the behavior of soil before and after initial liquefaction. All the material parameters needed for the material model used in this program can be determined from routine laboratory tests [72].

CUMLIQ is used for the evaluation of potential for liquefaction of a soil deposit using random vibration procedures. This tool estimates the potential for seismic liquefaction using known field and laboratory data of soils in combination with general statistical parameters of earthquakes. A check on the applicability of the method was made by recomputing the 34 cases of liquefaction and non-liquefaction reported by Seed and Peacock in 1970 [72].

CPTINT is a menu driven program for CPT and CPTU interpretation. More than 35 different correlation parameters including interpretation for soil classification type by friction and pore pressure ratios, relative density and internal friction by 3 methods, SPT-N and N_1 , cyclic stress ratio to cause liquefaction and that applied by earthquake, dynamic shear modulus, constrained modulus and Young's modulus over various depth intervals with a choice of units can be calculated in the parameter menu. New features include an enhanced level ground empirical liquefaction analysis, above ground water table, use of R_f or B_q zone for basis of interpretation in a separate liquefaction menu and input of user defined values of unit weight, SPT-N, water pressure, Q_t , R_f or B_q Zone, and CSR(EQ) with Depth [72].

LIQUITER determines by the Seed and Idriss in 1982, method the safety factor against liquefaction of saturated non-cohesive strata subjected to earthquake loading. Input data is the soil unit weight, SPT relative density and the median particle diameter together with the seismic parameters necessary to simulate the earthquake. The formula used enables one to take into account the maximum acceleration peaks through the ratio of cyclical stress induced by the earthquake with reference to an average value instead of the maximum value.

For each SPT test a correction factor is calculated that is a function of the depth where the test is performed and the relative density. It is possible to consider the presence of a surface load or the overlying strata. The analysis is based on the resistance factor to liquefaction calculated as the ratio between the limit shear stress that induces liquefaction and the maximum shear stress induced by the quake, taking into account the interstitial pressures and deformations developed during the earthquake [72].

Apollo is analyzing of potential liquefaction of soil layers for one-dimensional seepage. Apollo is incorporate a simplified procedure for 1D analysis of generation and dissipation of pore water pressures in sand deposit due to seismic excitation [72].

CLiq analyses liquefaction potential from CPT data. It provides users with a graphical environment specifically tailored for CPT and CPTU data. The software addresses advanced issues such as cyclic softening in clay-like soils and thin layer / layer transition zone detection.

It provides results and plots for each calculation step, starting with the basic CPT data interpretation through to final plots of factor of safety, liquefaction potential index and post-earthquake displacements, both vertical settlement and lateral displacements [72].

LiqIT carries out the analysis of liquefaction of loose saturated cohesionless soils under the effect of strong ground motion based on commonly used field data. It utilizes the most recent and well-known deterministic and probabilistic methods. Field data input from SPT, CPT and Vs measurements. Overall liquefaction potential is evaluated according to Iwasaki formulation [72].

FEQDrain is a finite element computer program for the analysis of the earthquake generation and dissipation of pore water pressure in layered sand with vertical drains. One method of soil stabilization for potentially liquefiable sites is the use of a system of vertical drains to dissipate the excess pore water pressure generated by earthquake loading, thus avoiding liquefaction. Performance assessments for these systems require the estimation of vertical drain spacing such that a maximum threshold level of excess pore pressure ratio is not exceeded. FEQDrain can be used to analyze three dimensional pore pressure generation and dissipation in layered sand deposits with geocomposite vertical drains for liquefaction mitigation. There are four different modes in which to analyze a problem.

Option 1 considers the performance of a soil profile without any ground improvement. Pore pressure generated by the earthquake can only migrate in a vertical direction. Option 2 examines soil layers with perfect drain that dissipates any pore pressure that reaches it. Option 3 examines soil layers with a drain, but with constant values of vertical and horizontal hydraulic conductivity within the drain. Option 4 examines soil layers with a drain with properties guiding the nonlinear resistance that water will feel as it flows into and through the drain. This option can also include the presence of a reservoir to collect water rising through the drain [72].

Peysanj is a suite of geotechnical program to carry out analysis of bearing capacity, liquefaction, plate load test, pressuremeter test, lateral earth pressure and retaining wall [72].

VERSAT-D2D is a computer program for dynamic 2D plane strain finite element analyses of earth structures subjected to dynamic loads from earthquakes, machine vibration, waves or ice actions. The dynamic analyses can be conducted using linear, or nonlinear, or nonlinear effective stress methods of analysis. The program can be used to study soil liquefaction, earthquake induced deformation and dynamic soil-structure interaction such as pile-supported bridges. It has the ability of evaluating effective stress model including dynamic pore water pressure and three models for computing dynamic pore water pressure; Modified stiffness parameters by dynamic pore water pressure; Calculation of ground deformations caused by soil liquefaction; Calculation of factor of safety against liquefaction [72].

SHAKE2000 is a software package that integrates ShakEdit and SHAKE. It provides a graphical user interface to SHAKE with the following features: Numerous attenuation relationships for estimating peak horizontal acceleration and velocity with distance; and, for the pseudo acceleration and pseudo velocity response spectra. Estimation of the cyclic resistance ratio (CRR) required to initiate liquefaction using: Standard Penetration Test results and the CRR vs. $N_{1,60,cs}$ chart developed by Seed et al. (1985) as revised in the NCEER (1997) workshop; and Cone Penetration Test (CPT) data, as recommended by Robertson and Wride. Calculation of settlement induced by earthquake shaking using either the Tokimatsu and Seed method in 1987, or the Ishihara and Yoshimine method in 1992 [72].

SoilGeophysical is an spreadsheet for geophysical and geotechnical analysis of bearing capacity, settlement analysis, simple slope stability, seismic hazard analysis, strong motion attenuation relationships, acceleration / displacement / velocity spectra, simple soil amplification analysis, soil liquefaction analysis [72].

SPTLIQ analyzes liquefaction potential and earthquake-induced ground settlements using standard penetration test (SPT) data based on the simplified Seed-Idriss procedures [72].

CPTLIQ analyzes liquefaction potential and earthquake-induced ground settlements using cone penetration test (CPT) data based on the simplified Seed-Idriss [72].

LATSPREAD estimates magnitude of anticipated liquefaction - induced lateral spreading based on empirical methods developed by Youd, Hansen, and Bartlett (1999) and Bardet, and Tobita in 1999 for user specified design earthquake [72].

GEOSTRESS predicts in-situ ground deformation response to both stress and thermal induced activities, and can model consolidation, seismic loading, and creep of frozen soil. It allows the user to prescribe initial temperatures, pressures, loads, displacements, and strains, and will permit construction sequence simulations for excavations, earth fills and structural loadings. The model features a coupled solution of transient pore pressure response, giving it the ability to model transient changes in effective stress, consolidation and seepage for complex engineering applications. The program also includes the use of post peak strain softening models that permit analysis of statically and dynamically induced liquefaction failures, and of slope instability caused by progressive failure [72].

Static Probing software records, presents and analyses the readout of static penetrometer probes with mechanical cone, electrical cone or piezocone. Parameters evaluated include: angle of friction; relative density; total overburden stress; equivalent N_{spt} value; shear deformation modulus; undrained cohesion; permeability K ; over consolidation ratio; consolidation coefficient C_v ; 1D consolidation modulus; Young's modulus; liquefaction potential [72].

LiquefyPro assists engineers in evaluating liquefaction potential and earthquake induced settlement. It is based on the most recent publications and conference proceedings to find commonly accepted and state of the art calculation procedures and offers multiple methods of calculation. Users can use a default method of calculation; or more advanced users can select an approach more suited to their local conditions. Plot of CRR, CSR, Factor of Safety, and settlement and plot of soil profile. Choice of SPT, CPT and BPT input data [72].

SWANDYNE is a dynamic interaction and nonlinear 2D program which uses fully coupled models. The program uses finite elements of both triangle and quadrilateral in shape. Since fluid velocity can be eliminated in differential equation stage so the resulting variables are skeleton displacement (u) and pore pressure (p). This is why it is called the u - p formulation. The program can be used to deal with Static, Consolidating, and Dynamic conditions under Draining or Undrained conditions. The program includes a library of constitutive models which can deal with monotonic to cyclic loading of sand and clay including the modeling of liquefaction behavior [72].

CyberQuake is used for determining seismic soil responses. It includes an extensible accelerogram database, tools for pre- and post-processing of seismic signals, graphics and online help. Nonlinear elastoplastic or equivalent linear computations may be carried out on multi-layered, dry or saturated soil profiles. The dynamic soil response is computed with the elastoplastic assumption for soil deposits. The main features of the soil behavior, such as secant shear modulus and damping ratio variation with respect to distortion, are reproduced by the cyclic constitutive model in a large strain range. Thus, it is possible to evaluate among other things: irreversible settlement, generation of sub-harmonics, saturation of the acceleration amplitude, coupling between various components of motion, and the liquefaction potential [72].

Dynamic Probing software records, presents and analyses dynamic penetrometer readings for any type of penetrometer including new or custom equipment, and borehole SPTs. The program contains a database of numerous correlations, compiled in an extensive literature search; it can then interpret the penetrometer readings, to yield a geotechnical parameter profile. Bearing capacity calculation for various foundation geometries, relative settlements, and design of piles and assessment of liquefaction potential in areas subject to seismic forces is provided [72].

ZSOIL is a geotechnical, foundation and underground flow engineering program based on the finite element method that simulates most of the natural processes encountered in geotechnical and foundation engineering including stability, consolidation, creep, excavation and underground flow and liquefaction [72].

DIANA is a well proven and tested software package with a reputation for handling difficult technical problems relating to design and assessment activities in concrete, steel, soil, rock and soil-structure interaction. The program's robust functionality includes extensive material, element and procedure libraries based on advanced database techniques, linear and non-linear capabilities, full 2D and 3D modeling and tools for CAD. DIANA offers also powerful solution procedures. Main functionalities include Direct and iterative solvers; Automatic load and time stepping; several incremental- iterative methods, Continuation methods and line search technique, Automatic substructuring [72].

The material models are Elasticity, Cracking models, Plasticity, Soil specials, Viscoelasticity, Interface non linear models, Temperature dependent material parameters, Viscoplasticity, Hyperelasticity, User-supplied material models.

Geotechnical applications often provide engineers with technically demanding challenges that can be advantageously solved with it. It offers full 3D modeling capabilities that can be used in applications as diverse as foundations, embankment, tunnels, excavations, mines and dams. In addition, since soil is multi-phase material, special procedures have been implemented to deal with pore pressure modeling in the soil. Advanced analysis capabilities are also available for ground water flow, consolidation, earthquake and liquefaction process that are essential for accurate prediction of those types of coupled problems [72].

Diana allows modeling of the full dynamic response of a system to be simulated either in the frequency domain or in the time domain. It also contains a specific library of material models for simulating the material behavior of liquefied soil: Towhata-Iai model, Nishi model and Ramberg-Osgood-Bowl model. In transient analysis you can optionally add a viscous contribution to the constitutive models for liquefaction analysis. This viscosity can be a constant value or a multi-linear function of the excess pore pressure ratio [72].

2.6.2 Finite difference based liquefaction analysis

FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) is a numerical modeling code for advanced geotechnical analysis of soil, rock, and structural support in three dimensions. It is used in analysis, testing, and design by geotechnical, civil, and mining engineers. It is designed to accommodate any kind of geotechnical engineering project where continuum analysis is necessary. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to the applied forces or boundary restraints. The explicit, Lagrangian calculation scheme and the mixed-discretization zoning technique used in it ensure that plastic collapse and flow are modeled very accurately. In comparison with similar software packages, it has several advantages which enhance its suitability for geotechnical problems. For instance, the “large-strain” calculation option in it allows simulations to capture significant deformation of the model up to, during, and after material failure [73].

In addition to its large strain option, the software facilitates the analysis of soil-structure interaction problems through the inclusion of specialized structural elements.

Several plasticity models in common use for geo-materials, such as modified Cam-clay and linear elastic Mohr - Coulomb, are already implemented in the code [73].

The geometry formed by the grid points and zones is called the grid and is analogous to a finite element mesh. Each zone (i,j) is defined by four grid points (i,j), (i,j+1), (i+1,j+1), and (i+1,j). Each zone is defined by four grid points but may take any quadrilateral shape. It divides most zones into four triangular subzones, although some zones may have only two subzones, depending on zone geometry [73].

Compared to the current numerical methods, the finite difference method has obvious advantages. Using the mixed discretization technology in FLAC3D calculation, the plastic damage or plastic flowing of materials is more precise and effectively simulated, so it is considered that the approach is more reasonable than that of numerical integration used in conventional finite element methods [74].

Dynamic motion equations are all used, even in the simulating of static problems. Therefore, it can be used to simulate the entire process of mechanical system from imbalance to balance.

It is not necessary to store a large stiffness matrix during the calculation in the explicit finite difference numerical method. Therefore, by using this method one can not only save computer memory then reduce computing time, but also enhance the speed of solving the problem. However, FLAC 3D has obvious shortcomings in Preprocessing, such as modeling and meshing.

Modeling can be only achieved by data files using FLAC 3D, which is not very intuitive, and cannot directly generate solid model. For more complex geological model, the detail data of control points modeling are required during modeling, which is easy to make mistakes and difficult to check. Modeling by FLAC 3D need to resolve the problems as follows: time-consuming and laborious. In order to obtain the direct result, three-dimensional simulation requires the long period of calculation.

The general calculation procedure in FLAC3D holds briefly as follows: first the equations of motion are invoked to derive new velocities and displacements from stresses and forces. Velocities are then utilized to derive strain rates, which in turn will yield new stresses. Cycling around loop described above lasts for one time step.

In every step within this loop, all of the grid variables are updated from known values that remain fixed while the step under consideration is done. Schematically the aforementioned are illustrated in Figure 2.27. The “box” which includes the constitutive equations takes the set of velocities already calculated and for each element, computes new stresses. The velocities are assumed to remain unchanged for the operation of the “box” namely the newly calculated stresses do not affect the velocities [73].

The geometry of the problems in FLAC3D is defined by grids and grid points. FLAC 3D organizes the grids and grid points in a row-and-column fashion. They are organized in horizontal direction, expressed in terms of “i” and in vertical direction, expressed in terms of “j”. The numbering of the rows and columns are started in the lower-left corner of the model [75].

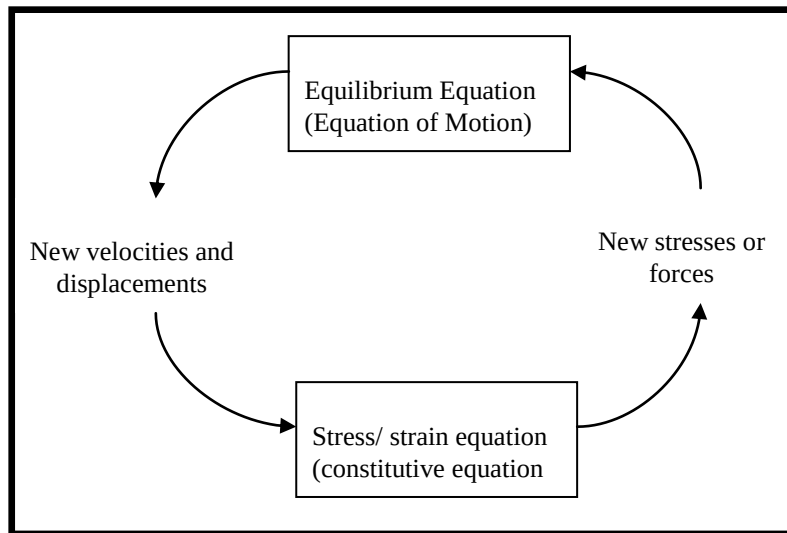


Figure 2.27 : Basic explicit calculation cycle in FLAC3D [73].

The first step in solving a problem is to divide the problem zone into a suitable grid (or mesh). The boundaries must be selected far enough [75].

Figure 2.28 shows a simple 4x3 grid. The x-y Cartesian coordinate system is used to reflect the coordinates of every point within the boundaries ABCD. Integers i and j (starting with 1 – not zero) are used to denote the grid lines (and elements) in two directions that are not necessarily orthogonal. Therefore, each element and node can be defined by a pair of i and j values. The dark line is defined by i = 3, the hatched element by i=3 & j=2, and the node (or grid point) E by i = 3 & j = 2 [75].

Essentially the model predicts the constitutive stress-strain behavior of the soil modeled based on an assumed hyperbolic relationship and thus the associated volumetric response of the soil skeleton is estimated using a flow rule that is a function of the current stress ratio. One of the model counts shear stress cycles by tracking the shear stress acting on horizontal planes while looking for stress reversals. The cyclic stress ratio (CSR) of each cycle is measured and this is used to compute the incremental "damage" which is translated into an increment of excess pore pressure [77].

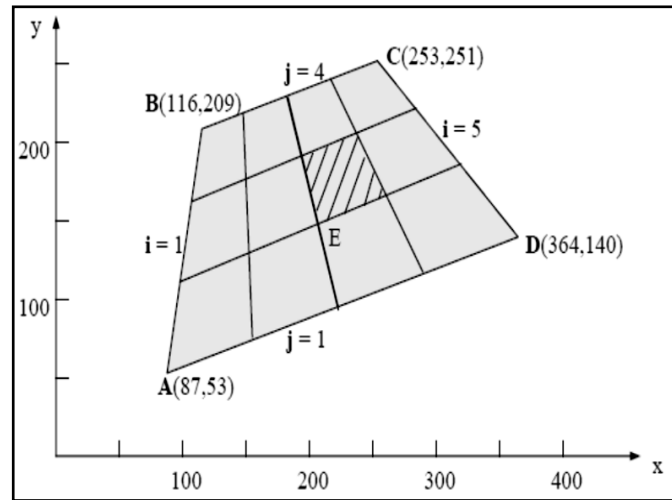


Figure 2.28 : Finite difference mesh in FLAC3D[75].

It is also described as a “decoupled” (or “loosely coupled”) effective stress model, because it generates pore pressure directly in response to the number of shear stress cycles required to trigger liquefaction. Pore pressures are generated incrementally in relation to cyclic strength of material as defined by a cyclic strength curve. According to several researchers, it has been described as “simple, robust and practice oriented”. One of its advantages is that its input parameters can be easily obtained from ordinary field investigations [77].

2.6.3 Neural network based liquefaction analysis

Neuroshell 2 is ANN model based software and it was adopted by many researchers in the field of geotechnical engineering in solving varieties of problems. Goh developed a back propagation neural network, BPNN, model to assess soil liquefaction [78,79].

He used eight soil and seismic parameters including standard penetration test value, mean grain size, fines content, equivalent dynamic shear stress, total and effective overburden stress, earthquake magnitude, and peak horizontal ground surface acceleration [78,79].

He reported that neural networks were proven to be feasible tools for soil liquefaction assessments, simpler to apply and yield more reliable results when compared to conventional methodologies. It was noted that Goh has used relatively small dataset in developing the BPNN model, which considered the main limitation of ANN approaches [79].

However, these models could be improved as new field case records became available. In his second neural network model for soil liquefaction potential evaluation, Goh introduced a new field test parameter, which is the cone penetration resistance, to the model and removed SPT-N value parameter. The most important parameter was considered to be cone penetration resistance. He concluded that his approach was simpler as no calibration and/or normalization was required to reach reliable output. It was also concluded that the performance of neural network model could be improved as further field case records became available [79].

In his further study, Goh developed PNN models to analyze the databases based on cone penetration tests and shear wave velocity data. In this research, soil particle-size information was introduced as an input variable and the liquefaction phenomenon was considered as a classification problem. Ural and Saka in 1998 developed a BPNN model based on liquefaction records of earthquakes from Japan, China, North and South America. In this research, a small number of case records were used in the developing stage and the model was limited to sandy soil type. Juang in 1999 developed two BPNN models using model-based data and actual field results to compare Olsen and Robertson methods for liquefaction potential evaluations. The Robertson method reported to be slightly more accurate and conservative; both methods are fairly accurate in predicting liquefaction resistance of sandy soils [79].

Barai and Agarwal in 2002 investigated Instance Based Learning, IBL, models to assess soil liquefaction potential. The IBL model is tested on cone penetration test dataset and reported to yield better results when compared to existing regression models. As a final note, the authors suggest to explore a hybrid model combining IBL and neural networks [79].

Juang in 2003 trained a neural network model to develop a CPT-based empirical equation defining the unknown boundary curve, also known as limit state function. It is reported that their method compares favorably to widely used existing methods, is simpler in formulation, is easier to apply, and applicable to wider range of soils. Hanna in 2004 explored the efficiency of group piles in cohesionless soil using artificial neural networks [79].

Utilizing a back propagation neural network model, pile group efficiency is addressed by introducing several governing parameters including the method of pile installation, soil condition, cap condition, type of loading, pile cross section, pile length/diameter ratio, pile spacing/diameter ratio, and pile arrangement. The outcome of the model provides correct prediction in more than 80 percent of the cases while the conventional methods provide correct prediction in 45 percent of the cases at maximum [79].

In the last two decades, Artificial Intelligence (AI) has been used in several applications in civil engineering, because of their heuristic problem-solving capabilities.

Artificial neural network (ANN) is one of the AI approaches that can be classified as “machine learning”. It has the ability to simulate the learning capabilities of the human brain by automating the process of knowledge acquisition and data mining. The ANN is a collection of interconnected computational elements called neurons that have performance characteristics similar to the biological neurons. This brain like structure makes ANN models superior to knowledge-based models and mathematical formulae in making predictions that involve intuitive judgment and possess high degrees of non-linearity [79].

Basically, interconnections among neurons are established by weights, which are applied to all values passing through one neuron to another. Changing weights improve adaptabilities and prediction capabilities of the Neural Networks.

Neural Networks are arranged in three or more layers according to their connection to the outside world. Neurons receiving data are in the input layer; neurons carrying the results are in the output layer, and other neurons are in the hidden layers. Through the learning process, input and output data of a specific engineering problem are given and aforementioned weights among neurons are updated without requiring human development of algorithms [79].

Based on this method the software called NeuroShell 2 is generally used. It is a neural network product targeted towards computer science instructors and students. It contains classic algorithms and architectures. The NeuroShell 2 Advanced Options screen displays the independent modules that may be used to create a neural network application (Figure 2.29) [80].

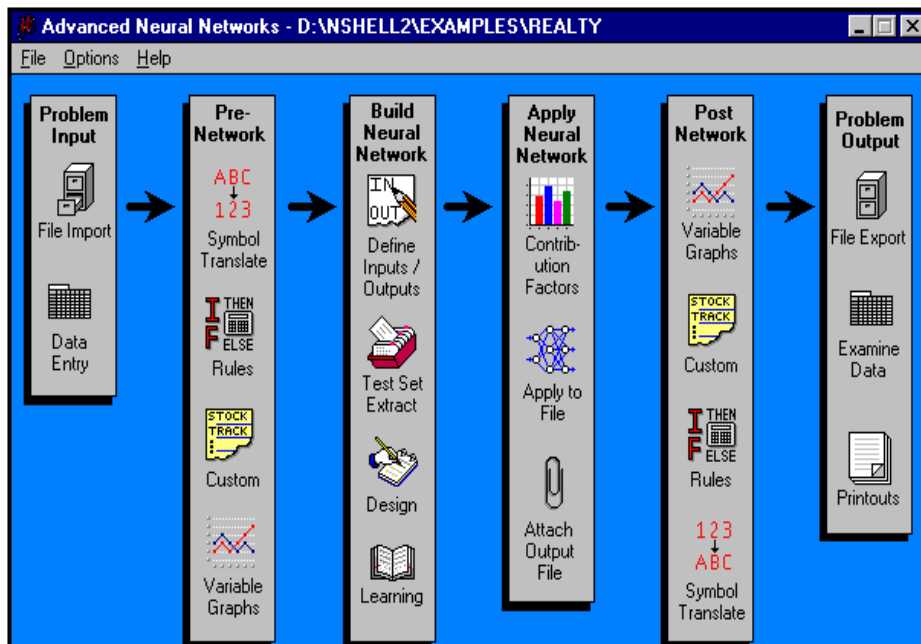


Figure 2.29 : Advanced options screen in Neuroshell2 [80].

The Advanced System it gives experienced neural network users the ability to create and execute 16 different neural network architectures, with more user control compared to the Beginner's System. Users can choose to enter their data in their familiar spreadsheet program or work in data grid. The program uses spreadsheet files as its internal format so you can view or edit them yourself. The Rules Module allows us to create If / Then / Else rules to pre-process data. We may also use the Rules Module to post-process the network's predictions and classifications. It gives us great flexibility in choosing which columns of your spreadsheet are inputs and which are outputs (Figure 2.30) [80].

Define Inputs/Outputs - D:\NSHELL2\EXAMPLES\REALTY.PAT							
File Edit Settings Help							
Variable Type Selection Input							
Variable Name	bedrms	other rms	baths	sq.ft.	eat/kitch	acres	price
Variable Type	I	I	I	I	I	I	A
Min:	2	3	1	800	0	.25	85000
Max:	5	5	4	3600	1	5	345000
Mean	3.425	3.9	2.25	1896.25	.625	.83325	158012.5
Std. Deviation	.7120754	.7089176	.8164966	656.5633	.4902903	1.056233	63831

Figure 2.30 : Rules module in Neuroshell2 [80].

The Design Module offers a palette of 16 different network architectures/paradigms for different types of data. Design Module offers 16 "icon selectable" architectures, each with several learning methods Back Propagation architecture design modules are given in Figure 2.31 [80].

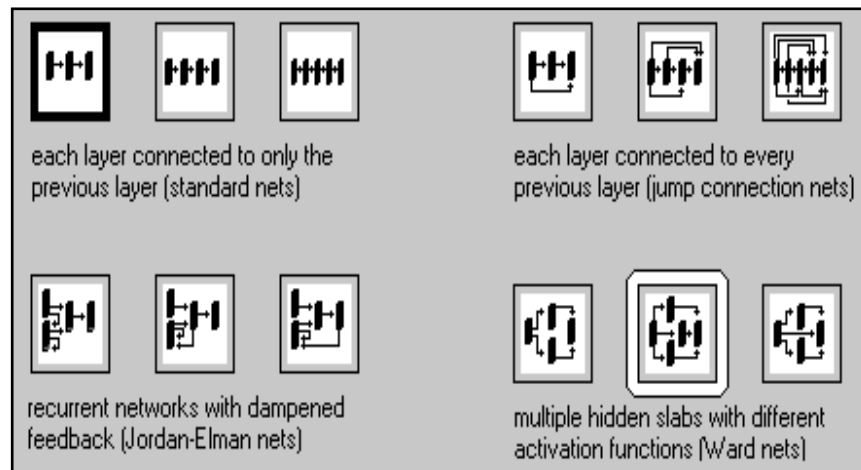


Figure 2.31 : Back propagation architecture design module in Neuroshell2 [80].

Each network architectures allows us to click on slabs of neurons and links to set parameters (Figure 2.32) [80].

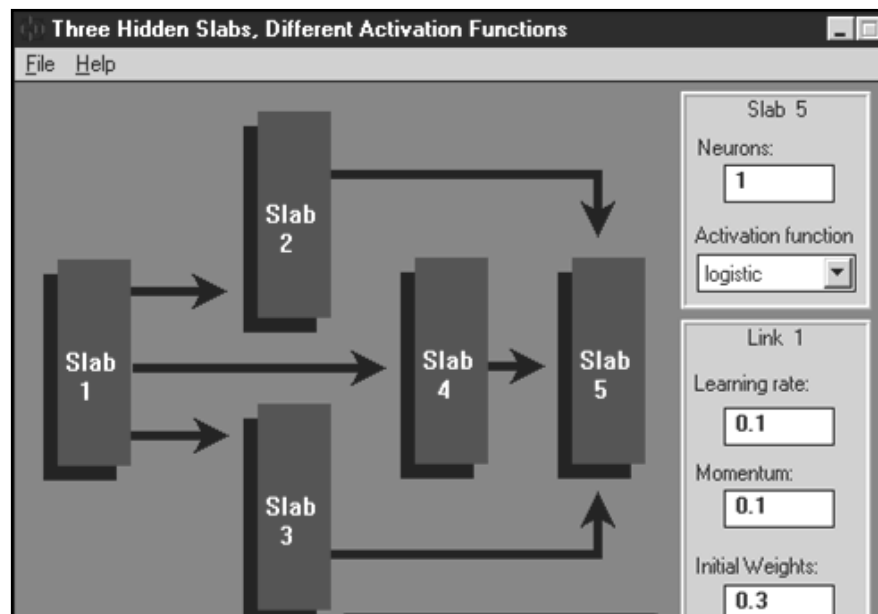


Figure 2.32 : Network architecture [80].

Twelve of the architectures include the Ward Systems Group version of back propagation, which has been enhanced for speed and accuracy. Standard Connections is the standard type of back propagation network in which every layer is connected or linked only to the previous layer [80].

Jump Connections is the type of back propagation network in which every layer is connected or linked to every previous layer. Recurrent Networks are a type of back propagation network is often used in predicting financial markets because recurrent networks can learn sequences [80].

Therefore, they are excellent for time series data. A regular feed forward network responds to a given input pattern with exactly the same output every time the given input pattern is presented. A recurrent network may respond to the same input pattern differently at different times, depending upon the input patterns which have been presented previously. Recurrent networks build a long-term memory of internal neurons [80].

Ward Nets is a type of back propagation network is able to detect different features in the data with the use of multiple slabs of neurons in the hidden layer, each with a different activation function. Activation functions are functions used internally in neurons to "fire" the neurons. When you apply a different activation function to each slab in the hidden layer, the network can discover novel features in a single pattern processed through the network. All back propagation algorithms come with a choice of Vanilla, Momentum, or TurboProp weight updates. With Vanilla, a learning rate is applied to the weight updates but a momentum term is not. Momentum means that the weight updates not only include the change dictated by learning rate, but include a portion of the last weight change as well. All of the back propagation networks include the Calibration feature, which prevents overtraining and increases the network's ability to generalize well on new data [80].

The Kohonen Self Organizing Map network used in NeuroShell 2 is a type of unsupervised network, which means it has the ability to learn without being shown correct outputs in sample patterns. The networks are able to separate data into a specified number of categories [80].

Probabilistic Neural Networks (PNN) is known for their ability to train on sparse data sets and they train in only one pass of the training set. PNN separates data into a specified number of output categories. PNN networks are often able to function as soon as two training patterns are available, so training can be incremental [80].

Genetic Adaptive nets (GRNN) optimize architectures and select inputs. Like PNN networks, General Regression Neural Networks (GRNN) are known for their ability to train in only one pass of the training set using sparse data sets. Rather than categorizing data like PNN, however, GRNN applications are able to produce continuous valued outputs.

GRNN is especially useful for continuous function approximation, and can fit multidimensional surfaces through data. In our tests we found that GRNN responds much better than Back propagation to many types of problems [80].

NeuroShell 2 includes a very powerful architecture which is called Group Method of Data Handling (GMDH) or polynomial nets. GMDH nets derive a mathematical formula which is a nonlinear polynomial expression relating the values of the most important inputs to predict the output variable. In fact, the GMDH network is not like regular feed forward networks and was not originally represented as a network. The GMDH network is implemented with polynomial terms in the links and a genetic-like component to decide how many layers are built. The result of training at output layer can be represented as a polynomial function of all or some of the inputs [80].

GMDH works by building successive layers with complex links (or connections) that are the individual terms of a polynomial. These polynomial terms are created using linear and non-linear regression. The initial layer is simply the input layer. The first layer created is made by computing regressions of the input variables and then choosing the best ones. The second layer is created by computing regressions of the values in the first layer along with the input variables. Again, only the best are chosen by the algorithm. These are called survivors. This process continues until the net stops improving. The resulting network can be represented as a complex polynomial description of the model just like the ones statisticians use. You may view the formula, which contains the most significant input variables. In some respects, it is very much like using regression analysis, but it is far more powerful than regression analysis [80].

Calibration solves one of the most difficult problems for neural networks knowing when to stop training. The longer the network learns the training set, the closer the network gets to "memorizing" the training set. If you present the network with a pattern that was in the training set, network will be able to make a very accurate prediction. If, you present network with a pattern that was not in training set, network may not be able to "generalize" well on data it has not "seen" before if you have let problem learn too long. Calibration corrects this.

For back-propagation networks, Calibration saves the network at the point where it gives the most accurate answers for patterns outside the training set. For GRNN and PNN networks, Calibration finds the optimum smoothing factor, a parameter that is used when you apply the network. The Genetic Adaptive nets calibrate with the genetic algorithm [80].

2.7 Discussion

Based on the above literature review, it can be concluded that soil liquefaction techniques continue to develop and there are not any 100 % successful technique yet. The purpose of this literature research is to make the liquefaction phenomenon and its evaluation types understood for the case study.

3. CASE STUDY

3.1 General

The Kurbağlıdere region in Istanbul, Turkey seismicity and their geological data are considered for this study. For this purpose, the Istanbul microzonation project reports, compiled by Soil and Earthquake Investigation Department under Istanbul Metropolitan Municipality were taken into consideration, and the data for the Kurbağlıdere region are used in the models as the case study data. The Istanbul microzonation project details and used Kurbağlıdere region data are given in the sections below.

3.2 Turkey, Istanbul Microzonation Project

The 1999 Kocaeli Earthquake was responsible for 17,000 fatalities, 44,000 injuries, and left 250,000 homeless, caused \$25 to \$40 billion in economic loss [85]. Shortly after the August 17, 1999, Turkey (Kocaeli) Earthquake, the Istanbul Metropolitan Municipality area, especially the coastal areas had significant damage and destruction. At the request of the Government of Turkey, the quality of building construction, earthquake information, and the objectives of the various plans, needed to be revised [86]. The Istanbul Metropolitan Municipality carried out the project under the Department of Planning and Zoning. 1/5000 scale maps were prepared by the Soil and Earthquake Inspection Directorate. The directorate of Ministry of Public Works and Settlement General Directorate of Disaster Affairs prepared soil and seismic reviews of these studies in detail and shared the project results with the public and the relevant departments [86].

The primary objective of the Istanbul microzonation project is to develop the first of its multi-hazard mitigation planning manual for Turkey, in order to encourage municipalities to improve management of overall risk. As part of the project, Turkish legislation was also reviewed to identify impediments that prevent the implementation of multi-hazard mitigation planning.

The goal of this project is to develop a methodology on how the municipality can incorporate multi hazard microzonation and Risk Vulnerability Studies into the development planning process; and to implement the methodology to create disaster mitigation plans. The study discusses how to combine the natural hazards information for the purposes of generating multi-hazard high-hazard map. As a damaging earthquake is expected for Istanbul, disaster mitigation plans concentrate more on the subject of the earthquakes [85].

In this meaning, loss estimation studies illustrate how to estimate the number of damaged buildings and casualties that might be expected from an earthquake. Because of this purpose, the necessary datasets for the Kurbağlıdere region are taken from this microzonation project, and used in this thesis.

3.2.1 Data description

The field works of Istanbul for soil investigations has been carried out by the department in three stages which are; exploratory borings, in-situ tests and soil sampling. The scope of the Istanbul microzonation project is in the 2912 grid (250 m × 250 m) total depth of 2830 pieces of 30 m, which is the normal drilling, 27 deep drilling, 764 liquefaction drilling units, 608 landslide drilling units [86].

Besides SPT, also 663 CPT experiments have been performed throughout in the Istanbul region. 2762 seismic refraction units - REMI measurement, 2625 electrical resistivity measurement units, 201 PS logging test, at 30 points and in 20 km long seismic reflection array microtremor measurements, and geophysical studies were carried out within the scope [86].

Studies carried out in field by the department, supported by experiments conducted in laboratory. Laboratory experiments include more than 53.938 experiments including, sieve analysis, hydrometer analysis, determination of Atterberg limits, free pressure test and triaxial compression test. Contents of geological studies and their amounts done for Istanbul are given in Table 3.1 [86].

Determination of engineering properties of soils were attained by the drilling operations as well as laboratory experiments, including the SPT (disturbed sample), 2,312 units UD (undisturbed sample).

Table 3.1 : Content of geological studies done throughout Istanbul in the Istanbul microzonation project [86].

Name of study	Number of points	Total (m)
Normal drilling	2.830	86.840
Deep drilling	27	4.201
Liquefaction analysis of borings	764	12.334
Landslide analysis borings	608	18.144
Faults, alluvium, based on additional drilling.	135	4.754
CPT	663	8.769
Trench studies	2	-

The samples obtained during field studies, were evaluated and tested in the laboratory. The numbers of samples tested in the project are shown in Table 3.2. Contents of these tests and the amounts for Istanbul are shown in Table 3.3.

Table 3.2 : Total sample numbers of Istanbul microzonation project [86].

Natural water content	Grain Size Distribution		Atterberg Limits	Free Preassure	Triaxial Pressure	Consolidation	Swelling
	Seive Analysis	Hydro meter					
53,938	53,938	124	46,432	1,120	462	2,315	2,315

Table 3.3 : Laboratory test and experiments done throughout Istanbul in the Istanbul microzonation project [86].

Soil Mechanics Experiments		Rock Mechanics Experiments		Age Determination Tests	
Experiment Name	Number of Experiment	Experiment Name	Number of Experiment	Test Name	Number of Test
Water content	29.072	Physical index	242	Konodont	77
Atterberg limit	21.608	Triaxial compression	143	Ostracod	75
Seive analysis	29.071	Uniaxial compression	88	Palynology	23
Hydrometer test	6.867	Indirect tensile	149	Radiometric (C14)	8
Consolidation	1.339				
Swelling pressure	1.339				
Triaxial pressure	214				
Uniaxial pressure	572				
Direct shear	6				

In the Kurbağlıdere region, a total of 21 SPT log data and their laboratory results are used in the models as case study data. The SPT log points are given in Figure 3.1.

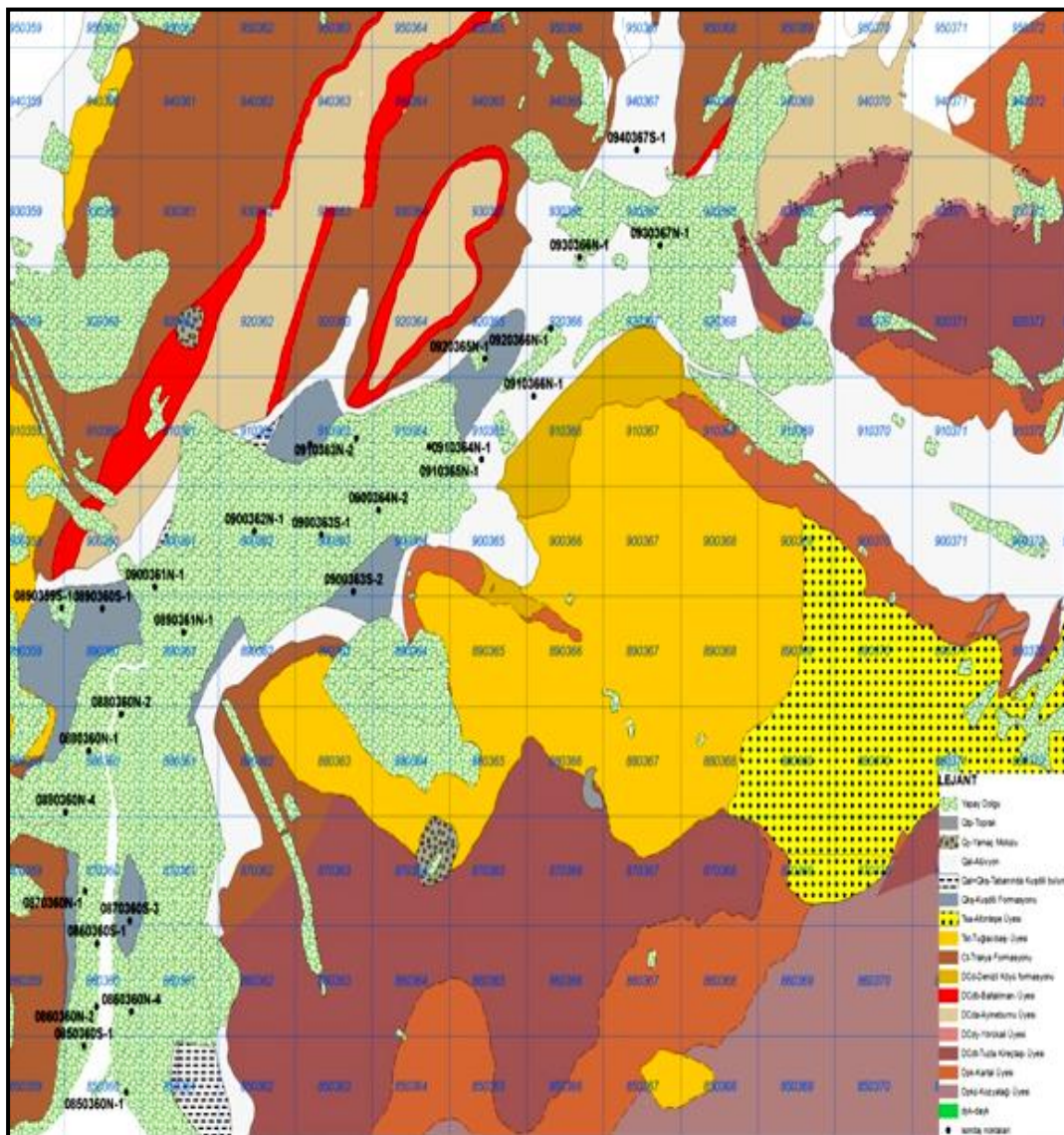


Figure 3.1 : SPT log information in Kurbağlıdere region [86].

An example of SPT log file which is in Zone 1 and also representing the point 1 is given in Figure 3.2.

3.2.2 Site characterization

The study areas of the Anatolian side of Istanbul contains Üsküdar, Kadıköy, Maltepe, Uskudar, Islands, Sancaktepe, Sultanbeyli, Maltepe, Kartal, Pendik, Tuzla districts and Cekmekoy's all sections of the basin and outside the forest area. Figure 3.3 depicts the three stages of the microzonation project of Istanbul and the districts in the region [86].

Location of the study area is Kurbağlıdere region at Kadıköy, Istanbul and is shown in Figure 3.4.



Figure 3.4 : Chosen case study area of Kurbağlıdere, Kadıköy, Istanbul[86].

The microzonation project of Istanbul area is defined by the following coordinates, given in Table 3.4.

Table 3.4 : Coordinates of the Istanbul microzonation project area [86].

European side of Istanbul	Area 1	X: 388,598.72	Y: 4,547,051.31
		X: 388,430.19	Y: 4,535,945.17
European side of Istanbul	Area 2	X: 407,375.58	Y: 4,535,681.75
		X: 415,860.39	Y: 4,541,134.16
Anatolian side of Istanbul	Area 3	X: 415,892.19	Y: 4,543,910.66
		X: 411,720.15	Y: 4,546,736.56

The total area of microzonation project Anatolian side of Istanbul is approximately 458 km². The project was completed in June 2009 with sharing reports and maps of 1/2000 scaled geology and engineering geology map, 1/5000 scaled hydrogeology map, 1/5000 scaled structural geology map, 1/1000 – 1/5000 scaled 3D slope map for determination of site risks, 1/5000 scaled ground shaking map, 1/2000 scaled liquefaction hazard map, 1/2000 scaled hazard map, 1/2000 scaled flooding map, 1/5000 scaled average shear wave velocity V_{s30} map, 1/2000 scaled site classification map and 1/2000 scaled land suitability map.

3.2.2.1 General soil properties of Kurbağlıdere region

The general soil formation and its soil properties are explained below from bottom to top of the soil.

- (a) Formation of Trakya (Palaeozoik, sandstone, and others)
- (b) The Ceylan Formation (Eocene, limestone, and others)
- (c) Gurpinar Formation (depending on Çekmece Formation) Oligocene - Miocene, sand, clay, clay stone, and many others.
- (d) Çukurçeşme Formation: Miocene, gravel and sand.
- (e) Gungoren Formation (depending on Çekmece Formation): Late Miocene, largely clay.
- (f) Bakirkoy Formation (Çekmece Formation): Late Miocene, limestone, marl.
- (g) Alluvium pockets and others: Mainly Holocene, clay, sand, beach sand, and topsoil
- (h) New Fills

3.2.2.2 Dynamic soil properties

Geotechnical investigations in Istanbul include thousands of boreholes. The boreholes with depths of 25m to 55m were drilled in the area. To analyze and evaluate the available geotechnical data, the area was divided into cells of 250m × 250m. For each cell representative soil profiles were determined based on detailed assessment of available geotechnical data.

Representative soil profiles for each cell where one or more borehole data were available were created by considering the most suitable borehole. For the cells with no available borehole information, representative soil profiles were selected from the neighboring cells by utilizing the available data. With these available data by using simplified equations method analysis, liquefaction hazard maps are prepared. In the coastal area, the alluvial silt and fills streams with high water content, but firmness of the ground is low. This area is in the first seismic zone, and in these areas, the risk of liquefaction is high.

In the areas within project, for the liquefaction potential is calculated by the cyclic stress ratio compared to a cyclic ground resistance ratio. An example liquefaction hazard map for a small part of Istanbul is given in Figure 3.5 Liquefaction potential risk map for case study area of Kurbağlıdere is given in Figure 3.6.

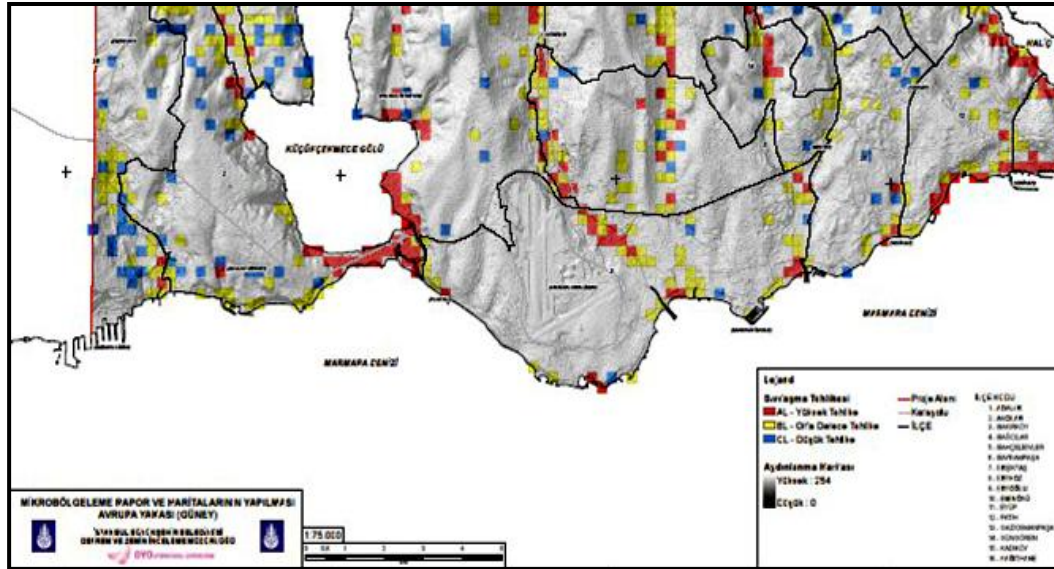


Figure 3.5 : Example map of liquefaction potential for Istanbul done by IBB [86].

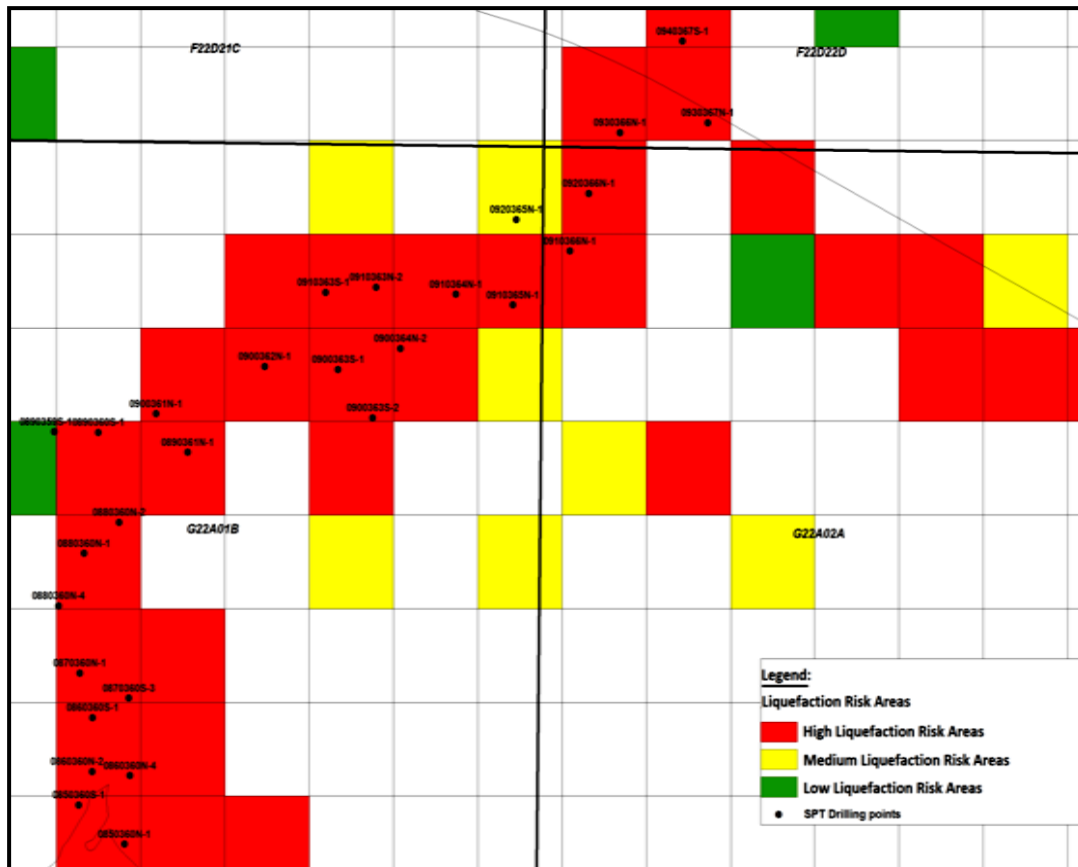


Figure 3.6 : Liquefaction potential map of case study area at Kurbağlıdere [86].

3.2.3 Earthquake information for Istanbul

On August 17, 1999 a magnitude $M_w=7.4$ earthquake struck the Kocaeli and Sakarya provinces in northwestern Turkey, a densely populated region to 40% of the industrial potential of the country. Three months after the Kocaeli earthquake, the M_w 7.2 Duzce earthquake took place to the immediate east of the fault rupture of the Kocaeli earthquake. Being associated with the ‘Duzce fault’, Duzce earthquake was interpreted as an aftershock of Kocaeli earthquake, as well as an independent event. Nevertheless, it completed rupture on this portion of North Anatolian fault leaving segments within Marmara sea unruptured.

August 17, 1999, Kocaeli earthquake produced right lateral surface displacements over a distance of about 120 km between the Hersek delta to the west of Golcuk on the west and Akyazi on the east. Measured displacements along the fault rupture were changing between 2.5 and 4.5 m. Maximum measured slip was measured as about 5 m at a distance of about 30 km to east of Golcuk. Duzce earthquake produced a surface fault rupture of approximately 40 km on Duzce fault between eastern terminus of Koceli Earthquake rupture at Eften Lake and Bolu Tunnels at Elmalik. The maximum right lateral offset was measured to be about 4 m. Rupture had a significant vertical component as well ranging between 10 and 150 cm [87].

Maximum MSK (Medvedev–Sponheuer–Karnik scale) intensity of the Kocaeli earthquake was X, essentially assigned on the basis of fault rupture and excessive ground deformations. No official intensity map exists for the Duzce earthquake essentially due to the difficulty in separating the effects of the two earthquakes from each other. According to the estimations, maximum MSK intensity in the Duzce earthquake was IX [87].

The earthquakes occurred in 1999 and the earthquake that the region see with a maximum magnitude of equal or higher than magnitude 5.0 given in Table 3.5.

As it is seen in Figure 3.7, the Kadıköy, Kurbağlıdere region is in the first order earthquake risk zone with potential liquefaction during earthquakes.

Table 3.5 : The earthquakes occurred in past at the case study region.

No	Earthquake	Date	M _w
1	Kocaeli	August 17, 1999	7.4
2	Düzce	November 12, 1999	7.1
3	Hendek-Akyazi	August 23, 2000	5.8
4	Izmit	September 13, 1999	5.8
5	Sapanca-Adapazarı	November 11, 1999	5.7
6	Akyazi-Adapazarı	August 17, 1999	5.5
7	Duzce aftershock	December 11, 1999	5.4
8	Duzce aftershock	December 11, 1999	5.2
9	Izmit	August 31, 1999	5.2
10	Northeast of Bolu	February 14, 2000	5.0
11	Duzce-Bolu	November 19, 1999	5.0
12	Kaynasli-Bolu	December 11, 1999	5.0
13	Hendek-Adapazarı	July 11, 1999	5.0
14	Marmara sea	September 20, 1999	5.0
15	Hendek-Adapazarı	August 22, 1999	5.0
16	Cınarcık-Yalova	August 19, 1999	5.0

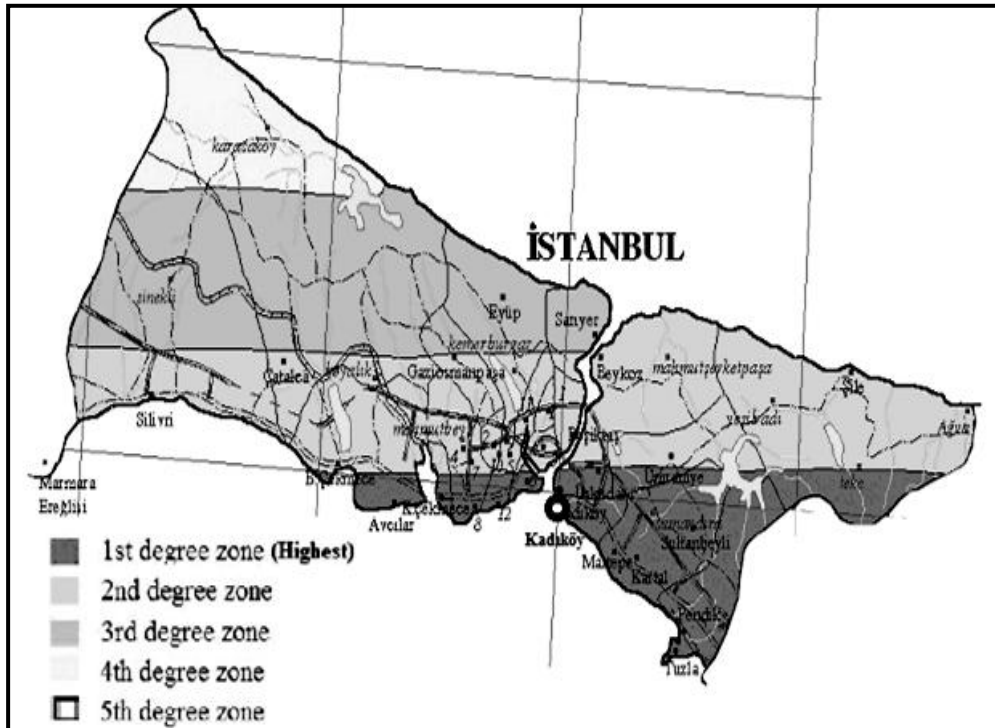


Figure 3.7 : Seismic risk map of Istanbul [85].

4. DEVELOPMENT OF MODELS

4.1 Finite Difference Model

4.1.1 General

In this chapter, the application of finite difference methods for liquefaction is discussed. There are different models introduced for finite difference method approaches. Data and case studies are given for finite difference models. Pore pressure generation for liquefaction due to ground acceleration will be examined by finite difference method based approaches. Within the scope of this chapter, the details of these numerical simulations of the results are presented with an example zone (zone 1) model's parameters and are given below.

4.1.2 Mathematical formulation

The theoretical background for finite difference method in FLAC3D is presented in this section. There are distinct differences between the two dimensional and three dimensional formulations, specifically in the 2D to 3D extension of the mathematical model that describes the mechanics of the system. This section describes the three dimensional formulation and its implementation in FLAC3D.

FLAC3D is an explicit finite difference program used to study, numerically, the mechanical behavior of a continuous three dimensional medium as it reaches equilibrium or steady plastic flow. The response observed derives from a particular mathematical model on one hand, and from a specific numerical implementation on the other. The mechanics of the medium are derived from general principles (definition of strain, laws of motion), and the use of constitutive equations defining the idealized material. The resulting mathematical expression is a set of partial differential equations, relating mechanical (stress) and kinematic (strain rate, velocity) variables, which are to be solved for particular geometries and properties, given specific boundary and initial conditions [73].

The method of solution in FLAC3D is characterized by the following three approaches: (1) Finite difference approach: First order space and time derivatives of a variable are approximated by finite differences, assuming linear variations of the variable over finite space and time intervals, respectively; (2) Discrete model approach: The continuous medium is replaced by a discrete equivalent – one in which all forces involved (applied and interactive) are concentrated at the nodes of a three dimensional mesh used in the medium representation; and (3) Dynamic solution approach: The inertial terms in the equations of motion are used as numerical means to reach the equilibrium state of the system under consideration. The laws of motion for the continuum are, by means of these approaches, transformed into discrete forms of Newton's law at the nodes. The resulting system of ordinary differential equations is then solved numerically using an explicit finite difference approach in time [73].

4.1.3 Soil generation

The soil generation of solution in FLAC3D is characterized by the following codes:

For both static analysis and dynamic analysis Mohr-Coulomb can be applied with these soil material parameters. These are (1) elastic bulk modulus (K), (2) cohesion (c), (3) mass density (ρ), (4) dilation angle (ψ), (5) internal angle of friction (ϕ), (6) elastic shear modulus (G), (7) tension limit (σ_t).

The second model used in dynamic analysis for sandy soils is Finn model and its soil material properties are given below.

These Finn (dynamic pore-pressure generation) model soil parameters are (1) elastic bulk modulus (K), (2) cohesion (c), (3) density mass density (ρ), (4) dilation angle (ψ), (5) ff c1 constant (C1), (6) ff c2 constant (C2), (7) ff c3 constant (C3), (8) ff c4 constant (C4), (9) ff switch = 0 for Martin et al. (1995) formula, and 1 for Byrne (1991) formula, (10) internal friction angle (ϕ), (11) elastic shear modulus (G) and (12) tension limit (σ_t).

The shear wave velocities are calculated with using (4.1) to find out the bulk and shear modulus parameters of soil layers [89].

$$V_s = 51.5 N^{0.516} \quad (4.1)$$

The maximum shear modulus, $G_{\max}$ is calculated for every layer with equation (4.2) and the bulk modulus, K for every layer is calculated by the equations (4.3 and 4.4).

$$G_{\max} = \rho * V_s^2 \quad (4.2)$$

where ρ is the soil density, and V_s is the shear wave velocity [75].

FLAC3D requires bulk modulus (K) and shear modulus (G) and they can be derived from E and ν from the equations of (4.3) and (4.4) [75].

$$G = \frac{E}{2(1+\nu)} \quad (4.3)$$

$$K = \frac{E}{3(1-2\nu)} \quad (4.4)$$

E and ν can be derived from G and K using equations (4.5) and (4.6).

$$E = \frac{9KG}{3K+G} \quad (4.5)$$

$$\nu = \frac{3K-2G}{3(3K+G)} \quad (4.6)$$

It is also necessary to input a density value here, and that should be the dry density of the material in the case of grids configured for fluid flow. When the grid is not configured for fluid flow (i.e., mechanical- only analysis, with pore water pressure values specified for the grids), we should specify dry density above the water table and saturated density below the water table [75].

Mohr-Coulomb model is a plastic model, where the material is assumed to behave elastically until yielding. The input parameters are: density, bulk modulus, shear modulus, friction angle, cohesion and tensile strength. The maximum possible value for the tensile strength is “ $c / \cot \phi$ ”, and the default is zero [75].

Table 4.1 presents sample of soil material parameter for zone 1 that is calculated by the above equations.

After, defining the soil properties and the soil geometry; the static analysis was performed. The static analysis is checked by using the maximum unbalanced force number that must reach $1*10^{-5}$. An example curve for maximum unbalanced forces to time graph is given in Figure 4.1.

Table 4.1 : The calculated soil property parameters for zone 1.

Soil Type	Depth (m)	Thickness (m)	SPT (N1)60	Shear, G (Pa)	Bulk, K (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion, C (Pa)	Dilation Angle (degree)
ArtificialFill	6	6	37	2.E+06	1.E+07	1800	20	2.E+03	0
GWT									
Sand	8	2	8	1.E+08	8.E+08	1800	35	0.E+00	10
SiltySand	12	4	15	2.E+07	6.E+08	1950	37	0.E+00	5
ClayeySand	22	10	10	9.E+07	5.E+08	1850	30	2.E+03	5
SandyClay	28	6	2	1.E+08	6.E+08	1850	30	2.E+03	5
Clay	39	11	3	1.E+08	5.E+08	1900	20	8.E+03	0
Sand	42	3	18	1.E+08	8.E+08	1800	35	0.E+00	10
SiltyClay	45	3	18	5.E+07	1.E+08	2000	27	5.E+03	0
Claystone	48	3	17	5.E+08	8.E+08	2200	25	1.E+04	0
Sandstone	50	2	17	3.E+08	1.E+09	2200	40	0.E+00	15
Rock	55	5	24	8.E+08	5.E+09	2800	46	1.E+07	12

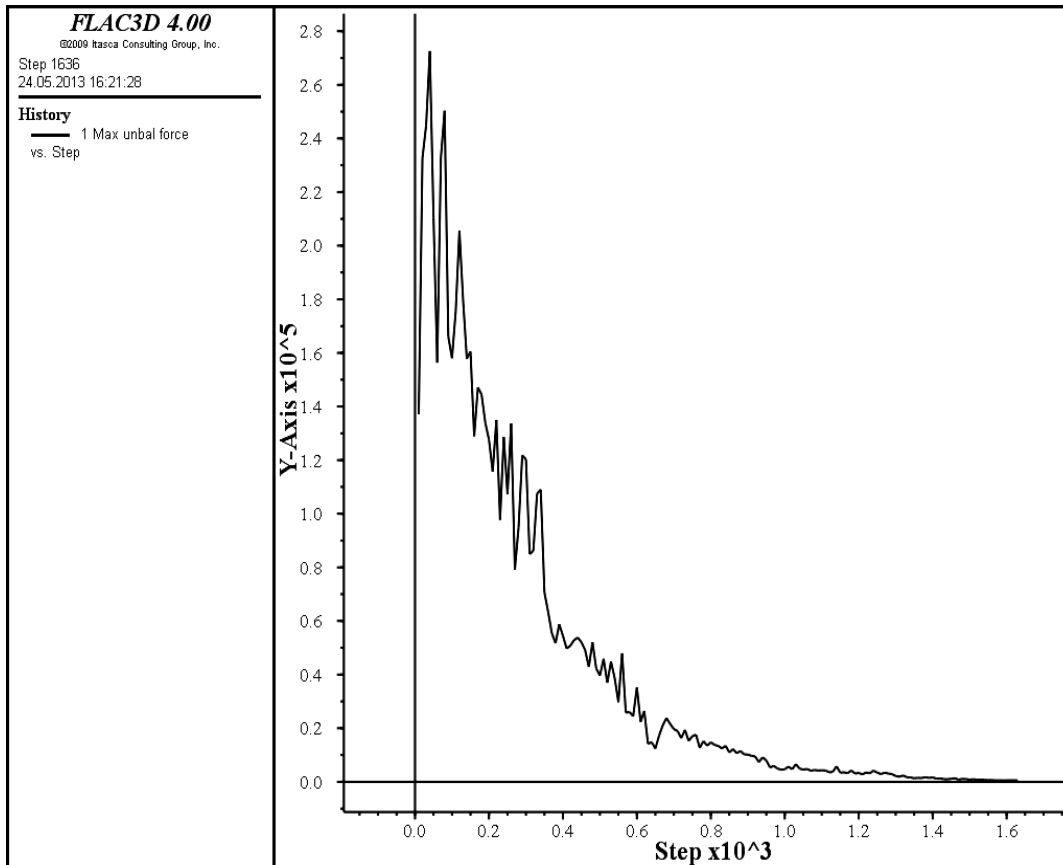


Figure 4.1 : An example maximum unbalanced force that reaches zero at static analysis.

Soil properties for all zones are defined for 250 m. to 250 m. areas (Figure 4.2) and then defined for 25 m. to 25 m. areas. An example soil property of layer for zone 1 is given in Figure 4.3.

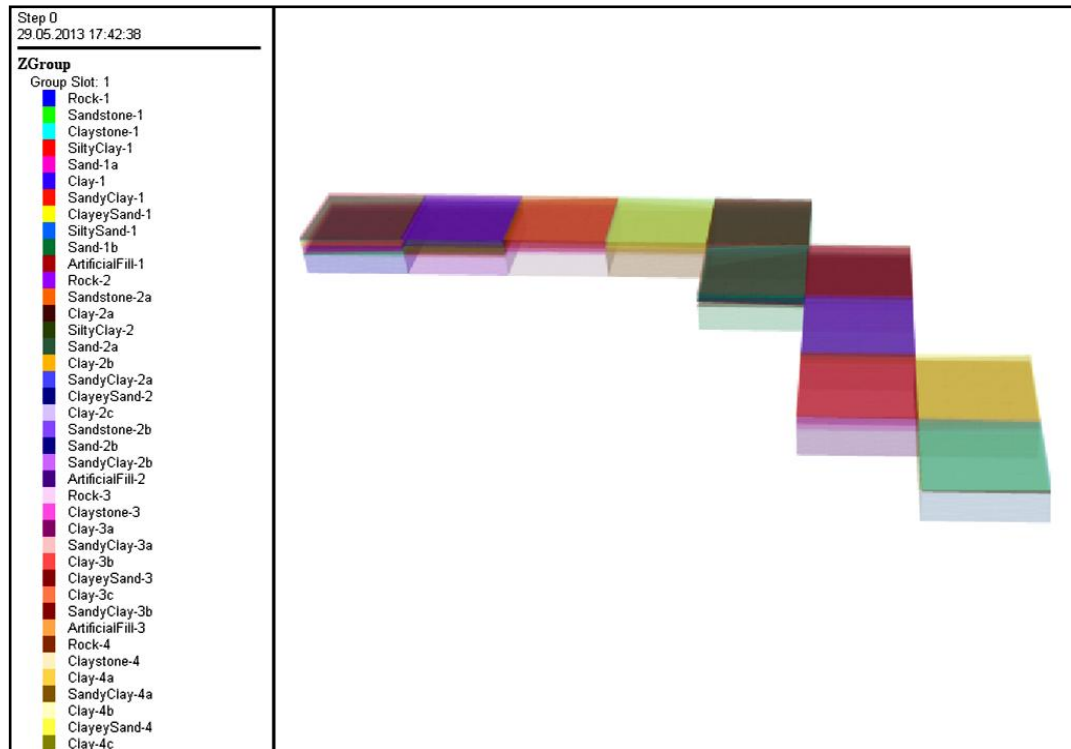


Figure 4.2 : Soil property groups defined for all 250m – 250 m zones for Kurbağlıdere region in FDM model.

The densities for all zones are defined and an example of defined densities for zone 1 is given in Figure 4.4.

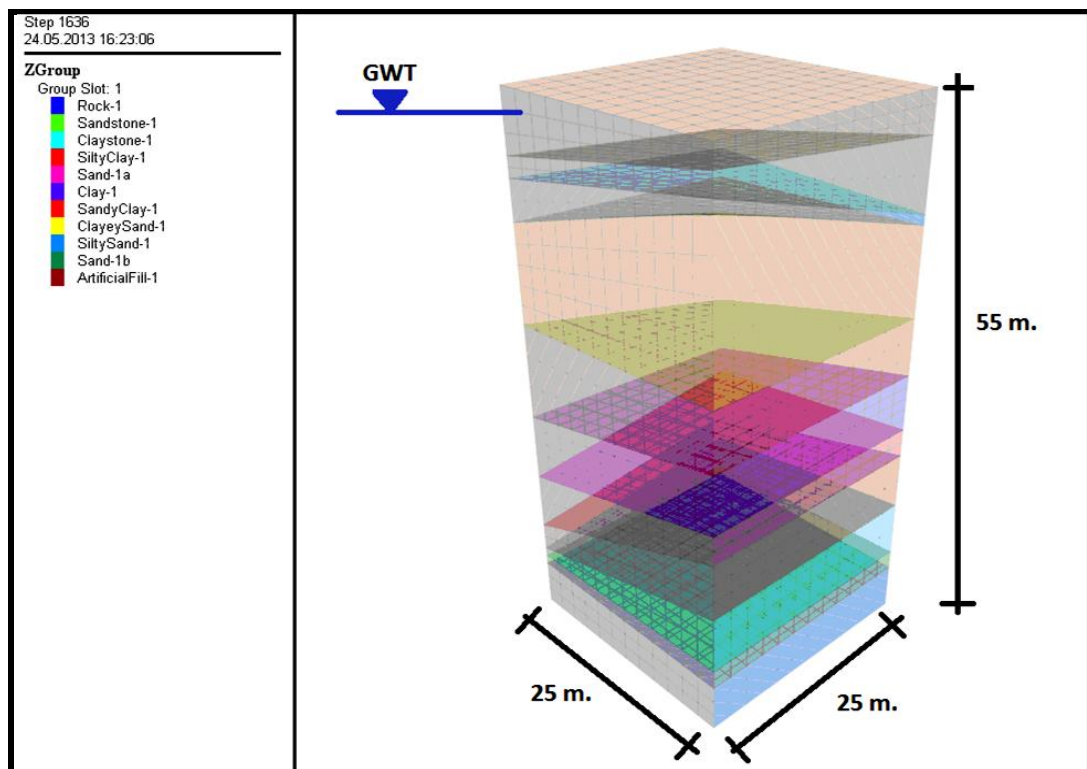


Figure 4.3 : Soil property groups for zone 1 in FDM model.

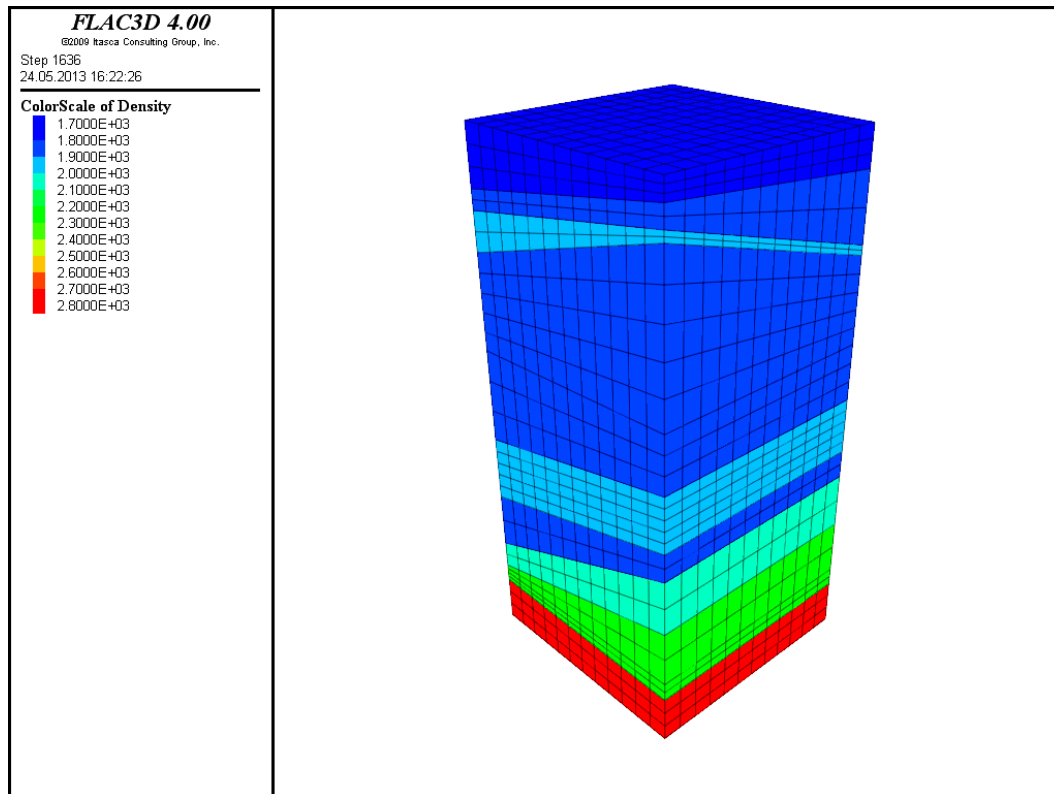


Figure 4.4 : Density values defined for soil layers at zone 1.

4.1.4 Grid generation

The 3D grid generation model includes advantages for solutions; on the other hand some difficulties can occur during modeling. Among three dimensional constant strain rate elements, tetrahedra have the advantage of not generating hour for model. However, when used in the framework of plasticity, these elements do not provide for enough modes of deformation. In particular situations, for example, they cannot deform individually without change of volume as required by certain important constitutive laws. In those cases, the elements are known to exhibit an overly stiff response compared to what is expected from theory. To overcome this problem, a process of mixed discretization is applied in FLAC3D. The principle of the mixed discretization technique is to give the element more volumetric flexibility by proper adjustment of the first invariant of the tetrahedra strain rate tensor. Application of the technique differs depending on whether primary discretization of the body is done into hexahedral zones or into tetrahedral. In the approach pioneered by Marti and Cundall, a coarser discretization in zones is superposed to tetrahedral discretization, and first strain rate invariant of a particular tetrahedron in a zone is evaluated as volumetric average value over all tetrahedral. Method is illustrated in Figure 4.5 [73].

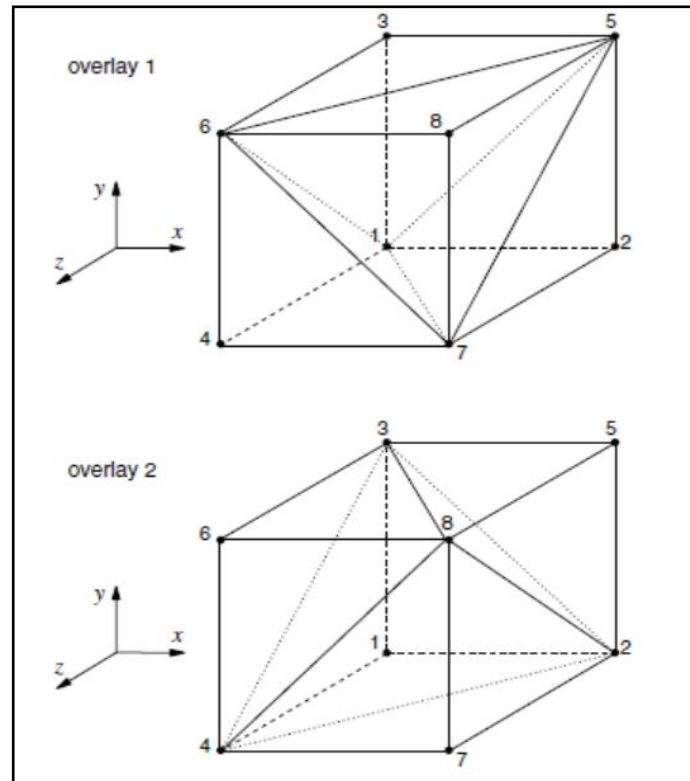


Figure 4.5 : An 8-node zone with 2 overlays of 5 tetrahedra in each overlay [73].

The area is divided to zones and the zonations given by numbers through in Figure 4.6 and the general mesh sizes are seen in Figure 4.7.

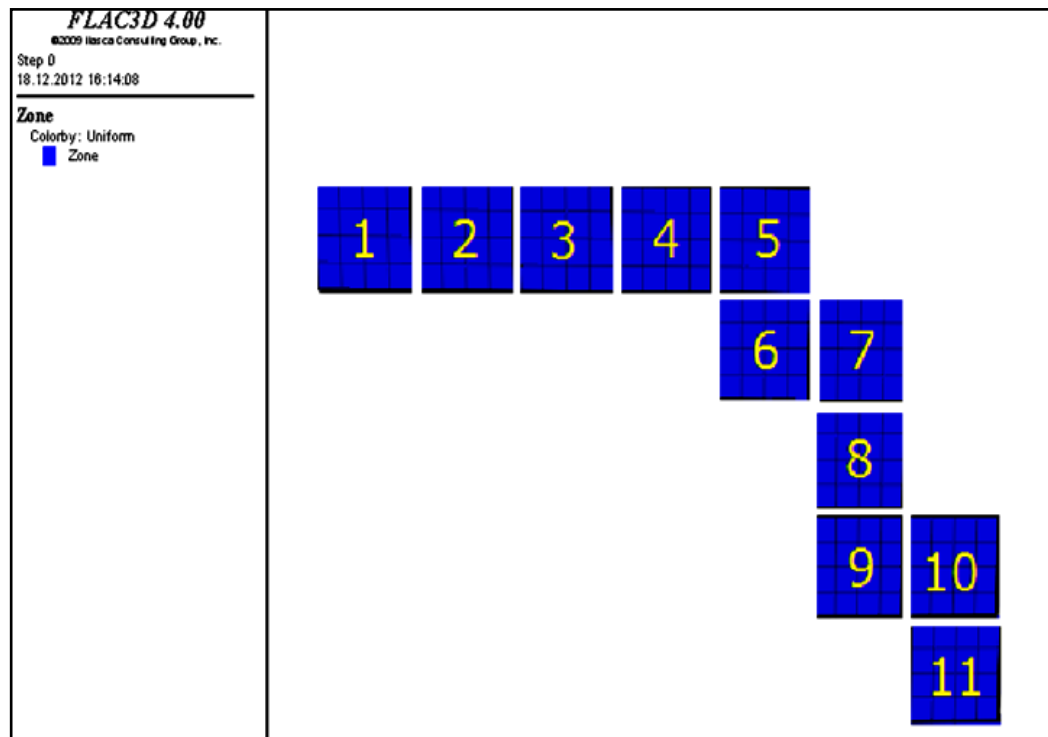


Figure 4.6 : Zonations of the Kurbağlıdere area in FDM model.

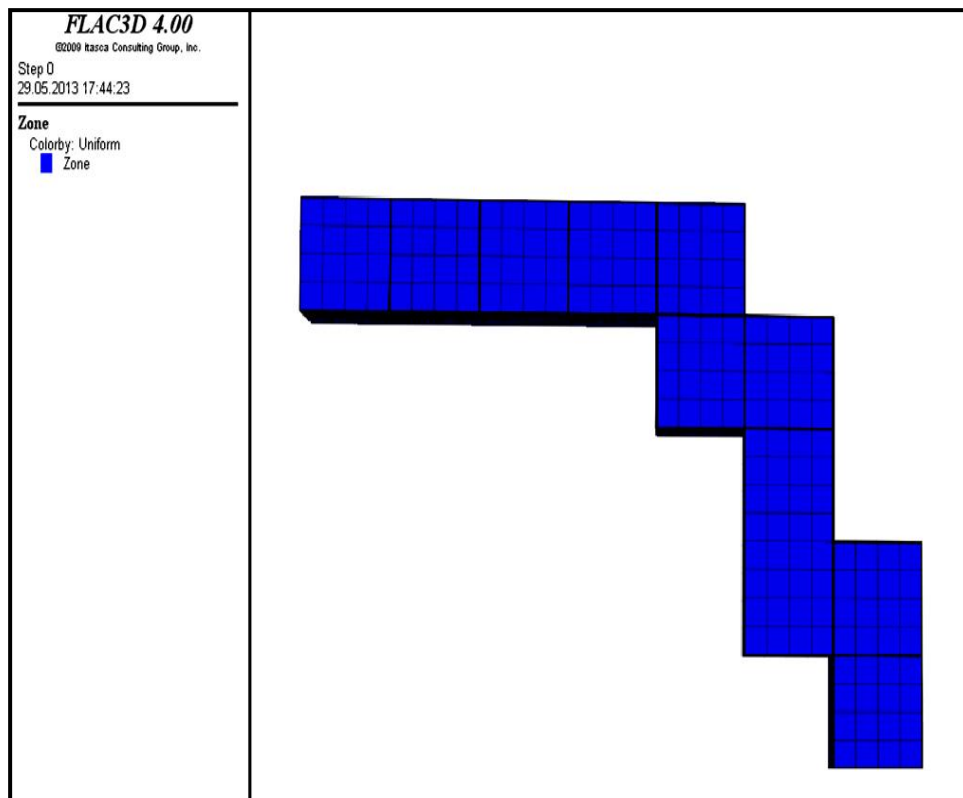


Figure 4.7 : Meshes created for all zones in FDM models.

The geometry of the area is created by using a 25m – 25 m mesh size, and an example mesh for zone 1 is given in Figure 4.8.

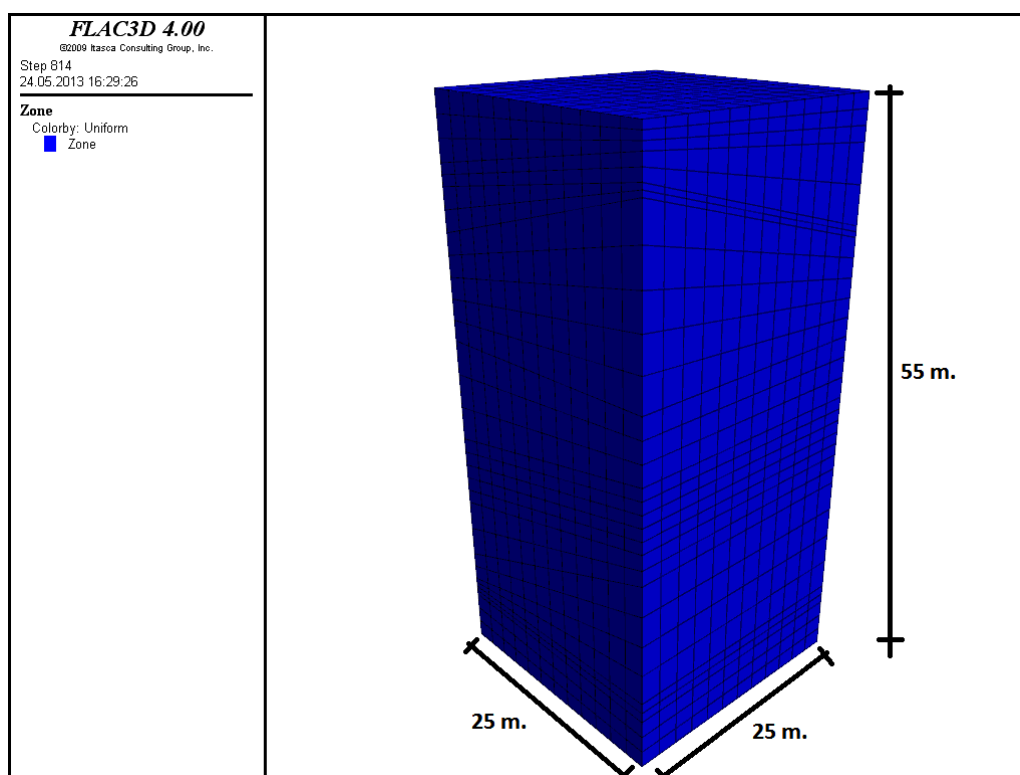


Figure 4.8 : Meshes created for zone 1 in FDM model.

Mesh sizes defined for all zones' models are given as with their coordinates by meters in Table 4.2.

Table 4.2 : Mesh sizes created for all zones in FDM models.

Mesh Sizes in Directions			
	x (m)	y (m)	z (m)
Zone 1	25	25	55
Zone 2	25	25	55
Zone 3	25	25	45
Zone 4	25	25	45
Zone 5	25	25	45
Zone 6	25	25	45
Zone 7	25	25	43
Zone 8	25	25	40
Zone 9	25	25	41
Zone 10	25	25	41
Zone 11	25	25	42

4.1.5 Boundary conditions

Model boundary conditions are prescribed in FLAC3D with the following keywords; “applies” for mechanical, fluid-flow and thermal conditions to any model boundary and “fix” allows velocity, pore pressure or temperature to be fixed or freed at selected grid points. With this command, velocity, pore pressure or temperature can be prevented from changing at selected grid points. If a fixed displacement is required, the velocities should be initialized to zero. The “free” command releases the grid point constraint set by the “fix” command. Free may be specified over a selected range. The range can be given in several forms. Boundary condition chosen for dynamic analysis for all zones is given in Figure 4.9.

In dynamic analyses, the application of boundaries to the model may cause the applied propagating waves to reflect back into the model. Using a larger model may minimize this problem but as a consequence, a large computational time becomes a problem. An alternative is to use “silent” (quiet or viscous or absorbing) boundary to overcome the problem. Silent boundary was suggested by Lysmer and Kuhlemeyer (1969). Silent boundary operates in the time domain and was based on the use of independent dashpots in the normal and shear directions applied at the model boundaries. FLAC states that a silent boundary is effective at absorbing the propagating waves for waves arriving at angles of incidence larger than 30°. Silent boundary is best suited for dynamic source applied within a grid [76].

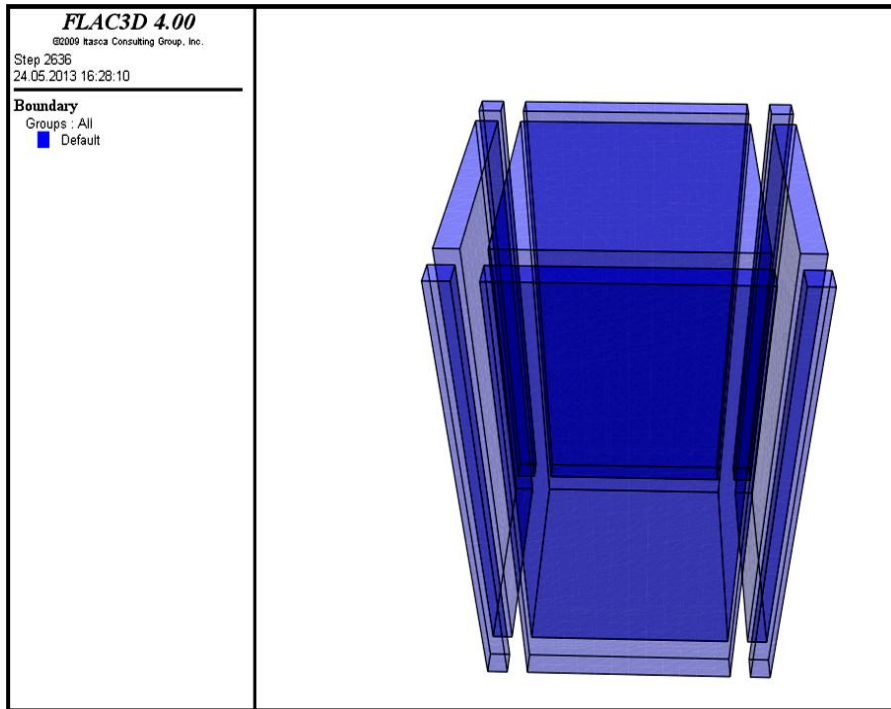


Figure 4.9 : Boundary conditions defined for all zones in the FDM models.

It should not be used along the side boundaries of a model when the dynamic source is applied at the top or bottom boundaries because the propagating wave will leak out of the side boundaries. For this case, free-field boundary should be used. The purpose of using free field boundary is similar to that of silent boundary in that it is used so that the outward waves propagating from inside the model can be properly absorbed by the side boundaries [76].

4.1.6 Initial conditions

In all civil or mining engineering projects, there is an in-situ state of stress in the ground, before any excavation or construction is started. In FLAC3D, an attempt is made to reproduce this in-situ state by setting initial conditions. Ideally, information about the initial state comes from field measurements.

However, when these are not available, the model can be run for a range of possible conditions. Although the range is potentially infinite, there are a number of constraining factors.

In a uniform layer of soil or rock with a free surface, the vertical stresses are usually equal to gpz , where g is the gravitational acceleration, p is the mass density of the material, and z is the depth below the surface.

However, the in-situ horizontal stresses are more difficult to estimate. There is a common – but erroneous – belief that there is some “natural” ratio between horizontal and vertical stress, given by $\nu/(1-\nu)$, where ν is the Poisson’s ratio. This formula is derived from the assumption that gravity is suddenly applied to a mass of elastic material in which lateral movement is prevented. This condition hardly ever applies in practice, due to repeated tectonic movements, material failure, overburden removal and locked-in stresses due to faulting and localization. Of course, if we had enough knowledge of the history of a particular volume of material, we might simulate the whole process numerically to arrive at the initial conditions for our planned engineering works, but this approach is not often feasible. Typically, a set of stresses is installed in the grid, and then FLAC3D is run until an equilibrium state is obtained. It is important to realize that there is infinite number of equilibrium states for any given system [76].

4.1.7 Constitutive model

There are thirteen basic constitutive models provided in FLAC3D, arranged into null, elastic and plastic model groups; the Mohr-Coulomb model is the conventional model used to represent shear failure in soils and rocks. Vermeer and deBorst (1984), for example, report laboratory test results for sand and concrete that match well with the Mohr - Coulomb criterion. In addition, there are two pore pressure generation material models available in the dynamic analysis option. These models are modified versions of the Mohr-Coulomb model that include cyclic volume change to simulate effects of liquefaction.

4.1.8 Earthquake motions

Design earthquake ground motions developed for seismic analyses are usually provided as outcrop motions, often rock outcrop motions. However, for FLAC3D analyses, seismic input must be applied at the base of the model rather than at the ground surface as illustrated in Figure 4.10. For a rigid base, a time history of acceleration (or velocity or displacement) is specified for gridpoints along the base of the mesh. While simple to use, a potential drawback of a rigid base is that the motion at the base of the model is completely prescribed. [73]

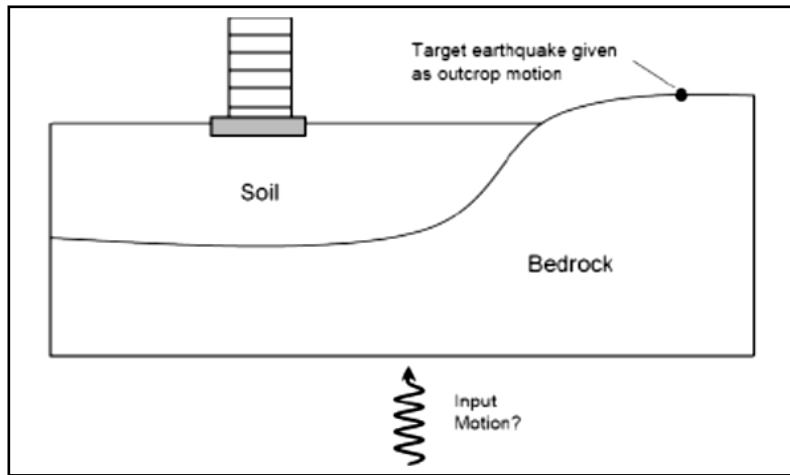


Figure 4.10 : Seismic input in FLAC3D [73].

On August 17, 1999, a magnitude $M_w = 7.4$ earthquake struck the Kocaeli area in the Northwest Turkey. The earthquake epicenter was located at Izmit along the strike-slip North Anatolian Fault. Details on the fault rupture mechanisms and strong motion recordings can be obtained from The Pacific Earthquake Engineering Research Center (PEER). The strong motion record used in these analyses (SKR) is obtained from a rock site, Sakarya Station. Maximum acceleration of the record is 0.38 g and the time step is 0.01 seconds. The strong ground motion station is located on a rock site in Sakarya. The acceleration characteristics of the record are given in Figure 4.11.

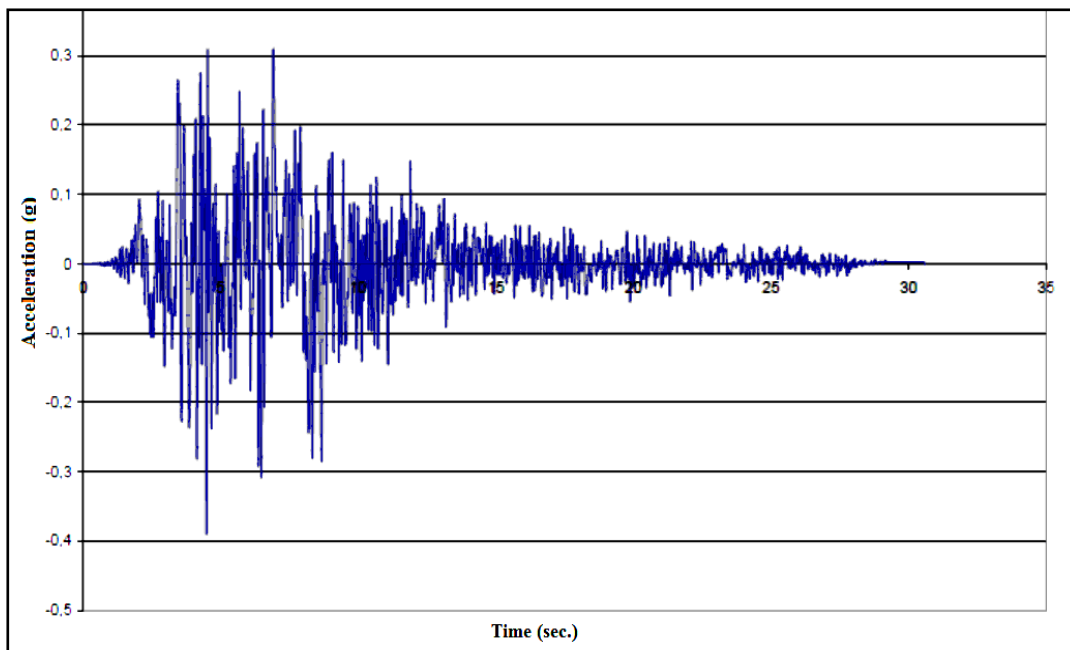


Figure 4.11 : Acceleration characteristics given as input motion to FLAC3D.

The duration of Sakarya (1999) earthquake was approximately 30 sec. which was very long for the analysis from run time point of view. Due to the runtime limitations, energy based approximation was implemented on Sakarya (1999) earthquake record which aims to eliminate the parts that do not contribute to the cumulative energy significantly. A 15 second motion is given for models and an example motion taken from in the layer of rock at zone 1 is given in Figure 4.12. The earthquake motion is given in one direction due to the FLAC3D software does not allow to model the earthquake motion in three directions at the same time. Because of this, the maximum acceleration record direction is chosen to define as earthquake strike direction.

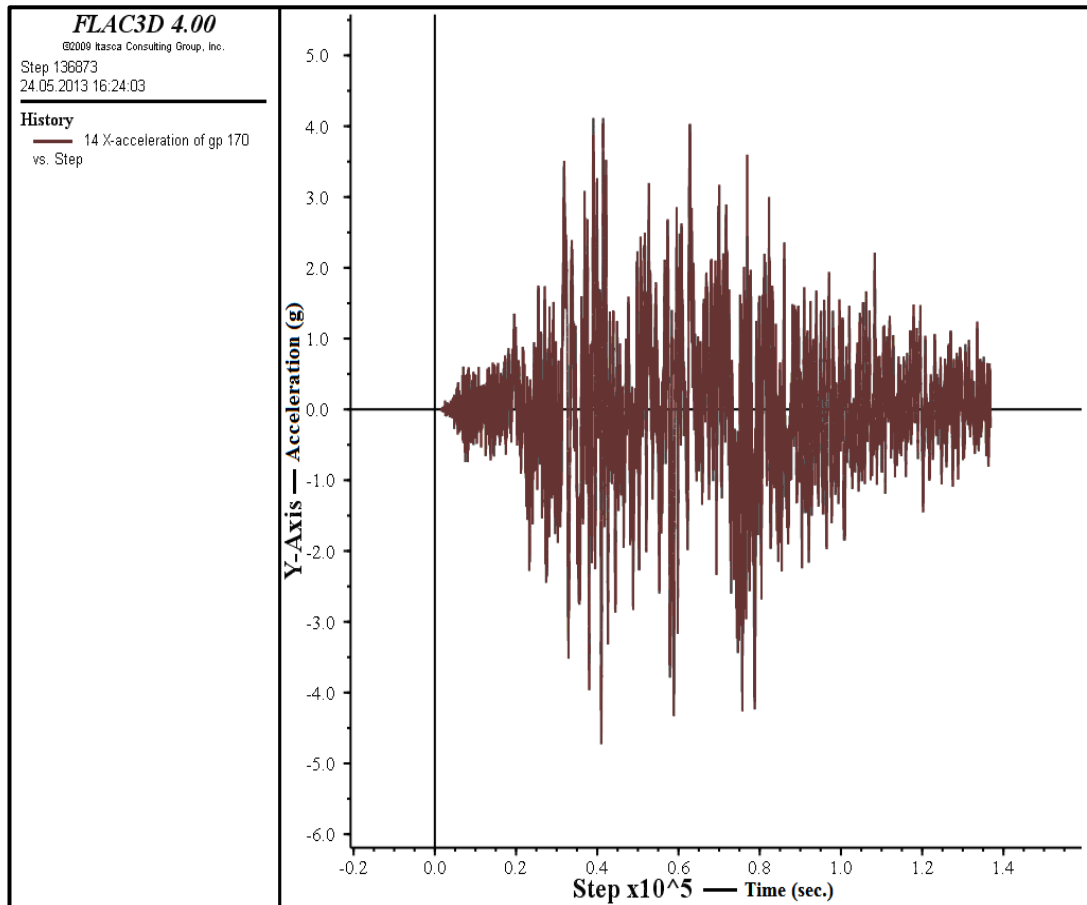


Figure 4.12 : Motion given in the layer of rock at zone 1 in FDM Model.

4.1.8.1 Filtering

For dynamic input with a high peak velocity and short rise time, the Kuhlemeyer and Lysmer requirement may necessitate a very fine spatial mesh and a corresponding small timestep. The consequence is that reasonable analyses may be prohibitively time and memory consuming [73].

In such cases, it may be possible to adjust the input by recognizing that most of the power for the input history is contained in lower frequency components. By filtering the history and removing high frequency components, a coarser mesh may be used without significantly affecting the results.

Damping should be applied to a system subjected to dynamic loading; otherwise, the system will oscillate indefinitely. Program has three options of damping embedded in the code, namely Rayleigh damping, local damping, and artificial viscosity. This section discusses Rayleigh damping because it is best suited for this research. Local damping is embodied in the program static solution and also used for dynamic analysis for as input time history for mostly all zones. Artificial viscosity is best used for waves with a sharp front. Rayleigh damping is frequency dependent and has two components: mass-proportional and stiffness proportional. The idea in FLAC3D is to adjust the center frequency ($f_{\min}$) of Rayleigh damping so it lies within the flat region, i.e. the range of predominant frequencies in the system. The center frequency ($f_{\min}$) is the frequency at which the mass-proportional and stiffness proportional damping each contributes one half of the total damping [76].

For many cases, the predominant frequencies are related to the natural mode of oscillation of a system.

$$f = \frac{C}{\lambda} \quad (4.7)$$

where:

f = the fundamental frequency related to the natural mode of oscillation of a system

C = the velocity of the wave related to the natural mode of oscillation of a system

= C_s or C_p , depending on the problem analyzed

λ = the longest wavelength related to the natural mode of oscillation of a system

Kuhlemeyer and Lysmer (1973) showed that the wavelength (λ) determines the accuracy for wave propagation problems. They found that the value of λ relates to the element length in the direction of propagation by a factor of one tenth to one eighth with one tenth factor suggested by FLAC3D using (4.8).

$$1 \leq \frac{\lambda}{10} \quad (4.8)$$

Earthquake time history is required to be applied in analysis. It may be applied as external dynamic loading at model boundary or as internal dynamic loading at internal gridpoints. The application of dynamic loading can be performed in four different ways; as an acceleration time history, a velocity time history, a stress time history, or a force time history. In our models, acceleration time history is defined.

4.1.8.2 Baseline correction

If a “raw” acceleration or velocity record from a site is used as a time history, the model may exhibit continuing velocity or residual displacements after the motion has finished. This arises from the fact that the integral of the complete time history may not be zero. Baseline correction usually applies only to complex wave forms derived, for example, from field measurements. When using a simple, synthetic wave form, it is easy to arrange the process of generating the synthetic wave form to ensure that the final displacement is zero. Normally, in seismic analysis, the input wave is an acceleration record. A baseline correction procedure can be used to force both the final velocity and displacement to be zero. For this reason, a baseline correction is also done for our model input motions.

4.1.9 Dynamic pore pressure generation

The basic theory of the Finn model is used in the dynamic models and the pore pressure and the total effective stresses at every layers middle point are evaluated. Then, the pore pressure ratio (r_u) is evaluated by dividing pore pressures to total stresses for all layers.

The Finn model was proposed by Martin, et al. (1975) using the following equation for the development of volumetric strains during cyclic loading by (4.9).

$$\Delta \varepsilon_{vd} = C_1(\gamma - C_2 \varepsilon_{vd}) + \frac{C_{3vd}^2}{\gamma + C_{4vd}} \quad (4.9)$$

where:

C_1, C_2, C_3, C_4 = constants

γ = shear strain

ε_{vd} = the volumetric strain

$\Delta \varepsilon_{vd}$ = the incremental volumetric strain

The value of the incremental volumetric strain ($\Delta\epsilon_{vd}$) is related to pore pressure (Δu) in one load cycle by the following equation (4.10).

$$\Delta u = \bar{E}_y \Delta\epsilon_{vd} \quad (4.10)$$

where:

Δu = increase in residual excess pore water pressure for the cycle

E = tangent modulus of the one-dimensional unloading curve at a point corresponding to the initial vertical effective stress (σ_{v0}')

Martin, et al. (1975) proposed that the value of the tangent modulus (E) could be determined with (4.11) as

$$\bar{E}_r = \frac{(\sigma_v')^{1-m}}{mk_2(\sigma_{v0}')^{n-m}} \quad (4.11)$$

where:

m , n , and k_2 = constants determined using at least three unloading curves in a consolidometer

σ_v' = the current vertical effective stress

σ_{v0}' = the initial vertical effective stress

Byrne (1991) used the data from tests performed by Martin, et al. (1975) and modified the Finn model and suggested use of the following simpler equation (4.12).

$$\frac{\Delta\epsilon_{vd}}{\gamma} = C_1 \exp\left(-C_2\left(\frac{\epsilon_{vd}}{\gamma}\right)\right) \quad (4.12)$$

The values of C_1 and C_2 are constants that can be estimated using the following empirical correlations with (4.13), (4.14) and (4.15).

$$C_1^c = 8.7 (N_1)_{60}^{-1.25} \quad \text{or} \quad C_1 = 7600 (D_r)^{-2.5} \quad (4.13)$$

$$C_2^c = \frac{0.4}{C_1} \quad (4.14)$$

where: $(N_1)_{60}$ = the corrected SPT blow count, D_r = the relative density of soil.

$$C_1 = 0.5 * C_1^c \quad \text{and} \quad C_2 = C_2^c \quad (4.15)$$

An alternative, and simpler, formula is proposed by Byrne (1991) to evaluate pore pressures and as it is explained in chapter 2 at the FDM literature the Finn constants (C_1 , C_2) are calculated for only sands or sandy soil layers. Finn constants used in Zone 1 are given in Table 4.3 and other zones' constants are given in Appendix C.

Table 4.3 : Calculated Finn coefficients for Zone 1 in FDM model.

ZONE 1 Used Constants						
Soil Type	Depth (m)	SPT - $N_{1,60}$	C_1^C	C_2^C	C_1	C_2
ArtificialFill	6	37	0.095	4.196	---	---
Sand	8	8	0.647	0.619	0.32	0.62
SiltySand	12	15	0.295	1.357	0.15	1.36
ClayeySand	22	10	0.489	0.818	0.24	0.82
SandyClay	28	2	3.658	0.109	1.83	0.11
Clay	39	3	2.204	0.182	---	---
Sand	42	18	0.235	1.705	0.12	1.70
SiltyClay	45	18	0.235	1.705	---	---
Claystone	48	17	0.252	1.587	---	---
Sandstone	50	17	0.252	1.587	0.13	1.59
Rock	55	24	0.164	2.442	---	---

Finn constants used in Zone 1 are given in Table 4.3 and other zones' constants are given in the Appendix C. The FDM based liquefaction analysis done by FLAC3D software due to the pore pressure generation analysis is summarized and given in Figure 4.13. These modelings are utilized on the references of previous stuies done with FLAC3D [90,91].

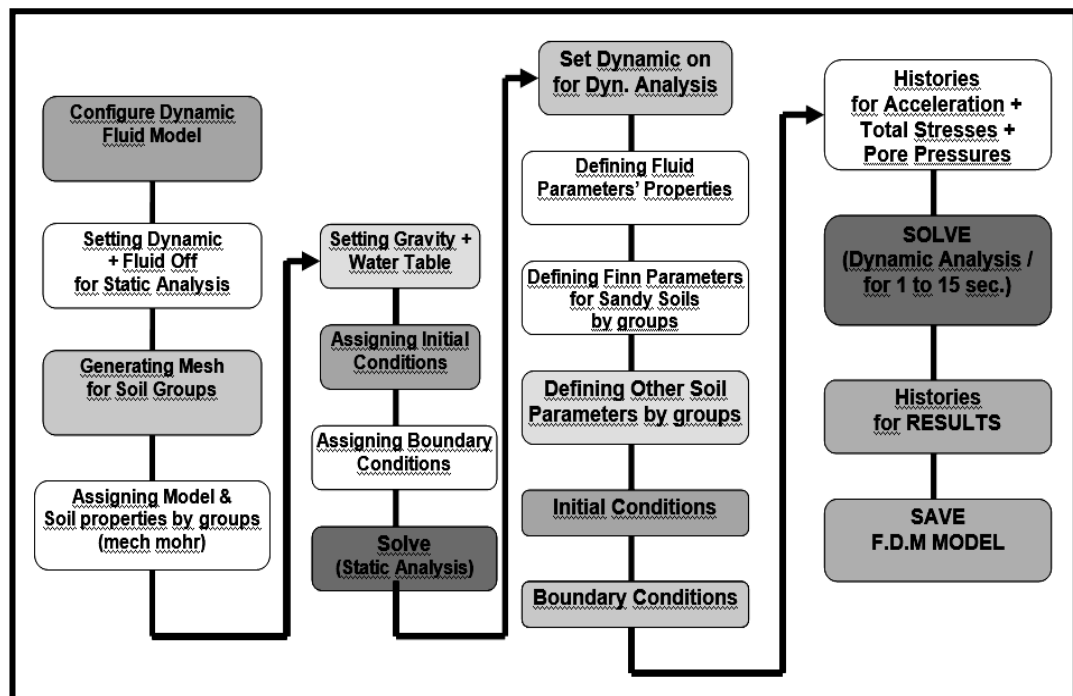


Figure 4.13 : Summary of FDM based liquefaction analysis done with FLAC3D.

4.2 Neural Network Model

4.2.1 General

In this chapter, application of neural networks for liquefaction is discussed. There are two models introduced for neural network approaches. Model 1 is Back- Propagation Neural Network (BPNN) approaches, and model 2 is General Regression Neural Network (GRNN) approaches. Data and case studies are given for Neural Network (NN). Factor of safety for liquefaction by means of pore pressure ratio (r_u) will be examined by NN approaches which are the BPNN and GRNN approaches. Values are obtained from FDM based model is given in Chapter 3. These values are used in these NN approaches and given below.

4.2.2 General modeling parameters

The proposed soil liquefaction potential models consist of two separate experimental datasets. As given in Table 4.4, case histories are taken from liquefied and non-liquefied layers from different zones. As presented in Table 4.5, these case records were divided randomly into testing, training, and validation datasets according to the layers.

Table 4.4 : Distribution of the data for liquefaction occurrence in NN model.

Database	Liquefied	Non-liquefied	Total Case Histories
SPT Database	45	465	510

Table 4.5 : Distribution of the data among phases in NN model.

Database	SPT Database	Percentage of Database
Training	352	%79
Testing	88	%17
Forecast	70	%13
Total	510	%100

4.2.2.1 Modeling of soil elements

Soil and seismic parameters characterizing soil type and material properties, seismic attenuation characteristics, magnitude and nature of loads, and other site conditions including stress, strain, strength, saturation and seismological aspects were selected and incorporated into the databases.

In this section, the soil elements used in modeling are given in Table 4.6.

Table 4.6 : Used soil parameters in NN model.

SPT Database Parameters	Abbreviations	Unit
Corrected SPT blowcount	(N1)60	---
Thickness of Layer	z	m
Shear Modulus	G	Pa
Bulk Modulus	K	Pa
Density	ρ	kg/m ³
Friction Angle	$\emptyset$	degree
Cohesion	c	Pa

4.2.2.2 Modeling of the earthquake motion

As the earthquake motion is taken consideration, it is assumed that only for a similar earthquake like 1999 earthquake will strike the region, so the magnitude of earthquake or the maximum acceleration is not used in the developed models as parameter considered (Table 4.7).

Table 4.7 : Earthquake motion parameters in NN model.

Database Parameters	Abbreviations
Magnitude	M
Max Acceleration	a _{max}

4.2.2.3 Modeling of the water

In this section, the water table is taken as a parameter for modeling and given in Table 4.8.

Table 4.8 : Water parameter in NN model.

Database Parameters	Abbreviations	Unit
Depth of ground water table	d _w	m

4.2.2.4 Datasets used in neural network models

Totally, 510 layer data are used in the neural network model and the dataset is given in the appendices. An example dataset used in the models is given as only for zone 1 at point 1 in Table 4.9. As it is seen from the table, the earthquake effect is chosen constant parameter.

Table 4.9 : Summary of SPT liquefaction case records in NN model.

	SPT - N60	Thick ness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density	Friction Angle (deg)	Cohesion (Pa)	Water Table (m)	Finn C1	Finn C2	a(max)	r _u
Artificial Fill-1	37	6	3.0E+06	5.0E+06	1700	20	2.0E+04	2.0	0.00	0.00	0.376	1.20
Sand-1b	8	2	5.0E+06	1.0E+08	1800	35	0.0E+00	2.0	0.32	0.62	0.376	1.00
Silty Sand-1	15	4	1.5E+07	3.3E+07	1950	30	0.0E+00	2.0	0.15	1.36	0.376	0.94
Clayey Sand-1	10	10	4.0E+07	8.0E+07	1850	28	2.0E+04	2.0	0.24	0.82	0.376	1.08
Sandy Clay-1	2	6	4.0E+07	8.0E+07	1850	30	1.5E+04	2.0	1.83	0.11	0.376	1.12
Clay-1	3	11	3.0E+07	6.0E+07	1900	22	3.0E+04	2.0	0.00	0.00	0.376	0.85
Sand-1a	18	3	5.0E+07	1.0E+08	1800	35	0.0E+00	2.0	0.12	1.70	0.376	0.72
Silty Clay-1	18	3	2.0E+07	4.0E+07	2000	25	2.0E+04	2.0	0.00	0.00	0.376	0.62
Clay stone-1	17	3	4.0E+07	8.0E+07	2200	35	5.0E+04	2.0	0.00	0.00	0.376	0.61
Sand stone-1	17	2	8.0E+07	1.6E+08	2200	40	0.0E+00	2.0	0.13	1.59	0.376	0.63
Rock-1	24	5	5.0E+08	1.0E+09	2800	42	1.0E+07	2.0	0.00	0.00	0.376	0.51

4.2.3 Architecture design

As it is discussed in previous chapters, in the last two decades, AI has been used in several applications in civil engineering, because of their heuristic problem-solving capabilities. ANN is one of the AI approaches that can be classified as “machine learning”. It has the ability to simulate the learning capabilities of the human brain by automating the process of knowledge acquisition and data mining.

Basically, interconnections among neurons are established by weights, which are applied to all values passing through one neuron to another. Changing weights improve adaptabilities and prediction capabilities of the Neural Networks. Neural Networks are arranged in three or more layers according to their connection to the outside world.

Neurons receiving data are in the input layer; neurons carrying the results are in the output layer, and other neurons are in the hidden layers. Through the learning process, input and output data of a specific engineering problem are given and above mentioned weights among neurons are updated without requiring human development of algorithms. Different algorithm and architectures are tested and the best two architectures are used in the models. Types of architectures are given in Figure 4.14.

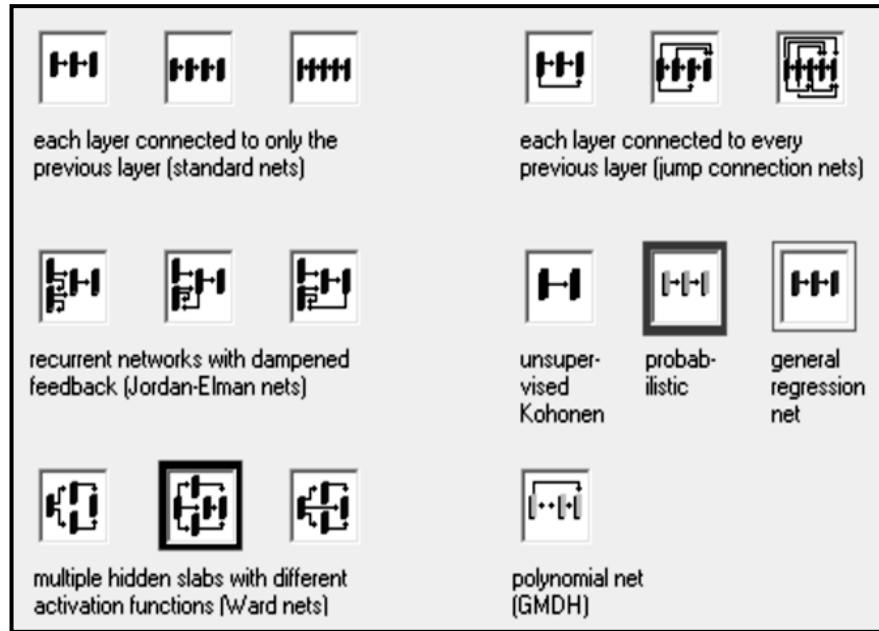


Figure 4.14 : Different architecture types tested for NN model.

Minimum, maximum, mean and standard deviation values of parameters are calculated and defined before the training phase starts as given in Table 4.10.

Table 4.10 : Variable types and minimum, maximum, mean and standard deviation values of parameters in NN.

Variable Name	SPT-N60	Thick-ness	Shear Modulus	Bulk Modulus	Density	Friction	Cohesion	Water Table	r_u
Variable Type	Input	Input	Input	Input	Input	Input	Input	Input	Output
Min	2	1.00	2000000	4000000	1700	20	0	1	0.48
Max	57	21.00	1.00E+10	2.00E+10	2800	42	1.00E+07	2.5	1.33
Mean	14	4.85	2.95E+08	5.90E+08	2003	29	6.20E+05	1.64	0.78
Std. Deviation	9	3.33	1.40E+09	2.79E+09	307	7	2.16E+06	0.49	0.17

4.2.4 Training phase

In this phase, the distance of the sample pattern from the given training set is measured. The training data is represented in N dimensional space, where N is the number of inputs introduced into the network.

4.2.5 Learning phase

In this phase, training data are presented into the network. Network's outcome in the training data is applied to the testing data and best smoothing factor for the network is explored. The value of the smoothing factor giving the smallest error is used in the final network. The objective is minimizing the mean squared error of the test set: therefore, amid presented test data, random testing datasets are generated in order to observe the network's performance.

4.2.6 Validation phase

In this phase, model accuracy and efficiency were examined by making prediction against case records, which were not used during model training and testing. In this phase, proposed algorithm does not require human development of proposed model and it rather confirms architecture's prediction capabilities of model. The hidden layers properties that are used in the model of the BPNN are given in Figure 4.15.

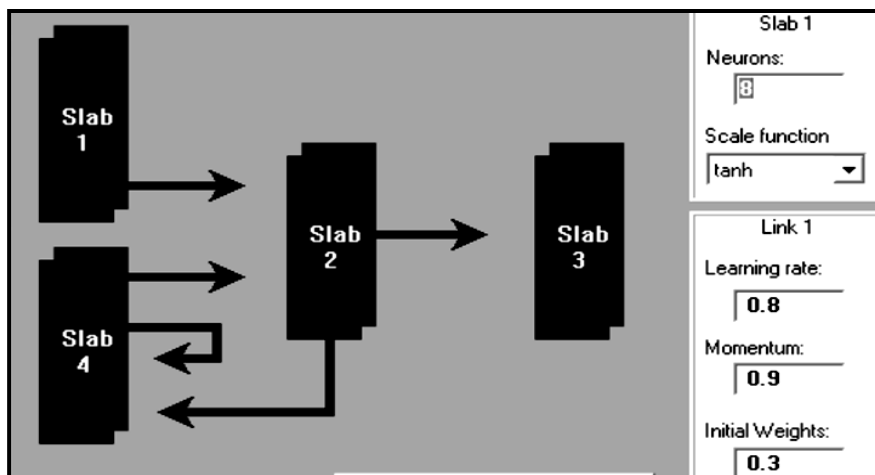


Figure 4.15 : An example hidden layer properties for BPNN architecture type.

The values that effects the models success rates are chosen and defined for model architecture of BPNN is given in Table 4.11. Hidden layers properties that are used in the GRNN model are given in Figure 4.16.

Table 4.11 : BPNN based model chosen layer parameter values.

Architecture Type	BPNN, Hidden layer dampened feedback link
Learning rate	0.8
Momentum	0.9
Initial Weights	0.3
Scale Function	tanh
Calibration number	100

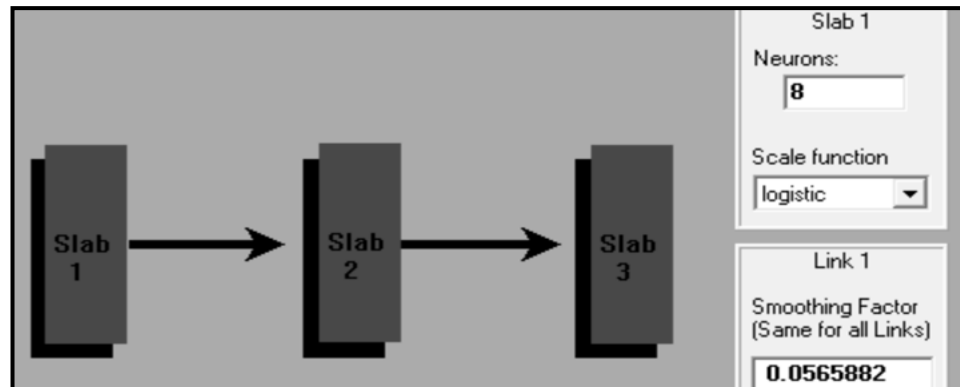


Figure 4.16 : An example hidden layer properties for GRNN architecture type.

The values that affect the models success rates are chosen and defined for GRNN architecture model too and these values are given in Table 4.12.

Table 4.12 : GRNN based model chosen layer parameter values.

Architecture Type	GRNN
Smoothing Factor	0.0565882
Scale Function	Logistic
Calibration Number	Genetic, adaptive
Distance Metric	City Block
Genetic Breeding Pool Size	20

NN based liquefaction analysis done by Neuroshell software due to the both BPNN and GRNN architectures is summarized and given in Figure 4.17. These two models are done by utilizing previous studies based on neural network algorithm [92-100].

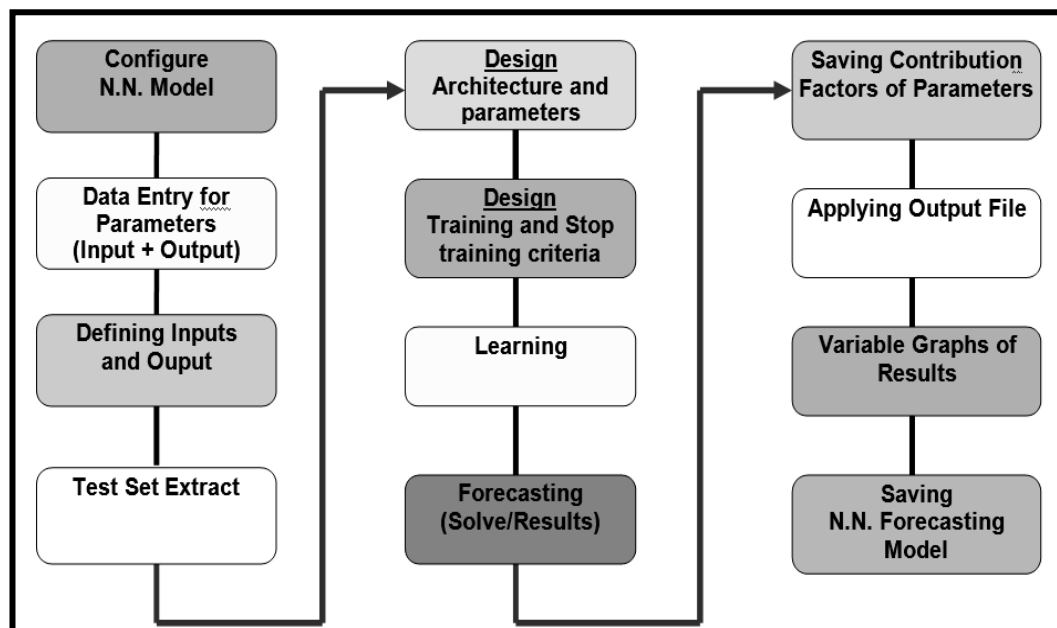


Figure 4.17 : Summary of NN based liquefaction analysis done with Neuroshell.

5. RESULTS

5.1 Introduction

In this chapter, the general results from FDM models and NN based models results are discussed. The FDM results are given in two phases. First phase is the results from zone 1 in the meaning of excess pore pressure and its pore pressure ratio for evaluating liquefaction potential of zone. The second phase is the comparison of 1D, 2D and 3D analysis types result differences in the meaning of liquefaction potential of point 1 for zone 1 and also for zone 8. For FDM models all zones results are given in the Appendix E. The neural network approaches models are also given in two phases. The first one is Model 1 with Back Propagation Neural Network (BPNN) approaches results, and the second one is model 2 with General Regression Neural Network (GRNN) approaches results. Only zone 10 and zone 11 layers' results which are forecasted zones are given in this chapter. All other zones results that are representing case study area (Kurbağlıdere region) are presented in appendices.

5.2 Kurbağlıdere Region FDM based Model Results

The pore pressure of layers for static analysis of zone 1 is given in Figure 5.1.

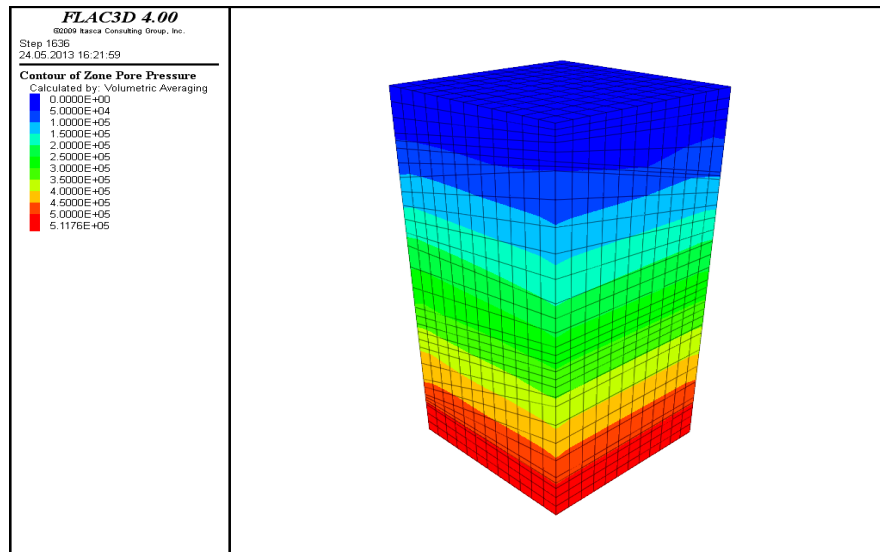


Figure 5.1 : The pore pressure values at layers in zone 1 in FDM static analysis.

The pore pressure of layers for zone 1 during the dynamic analysis is in the fifteenth second of the earthquake motion is given in Figure 5.2.

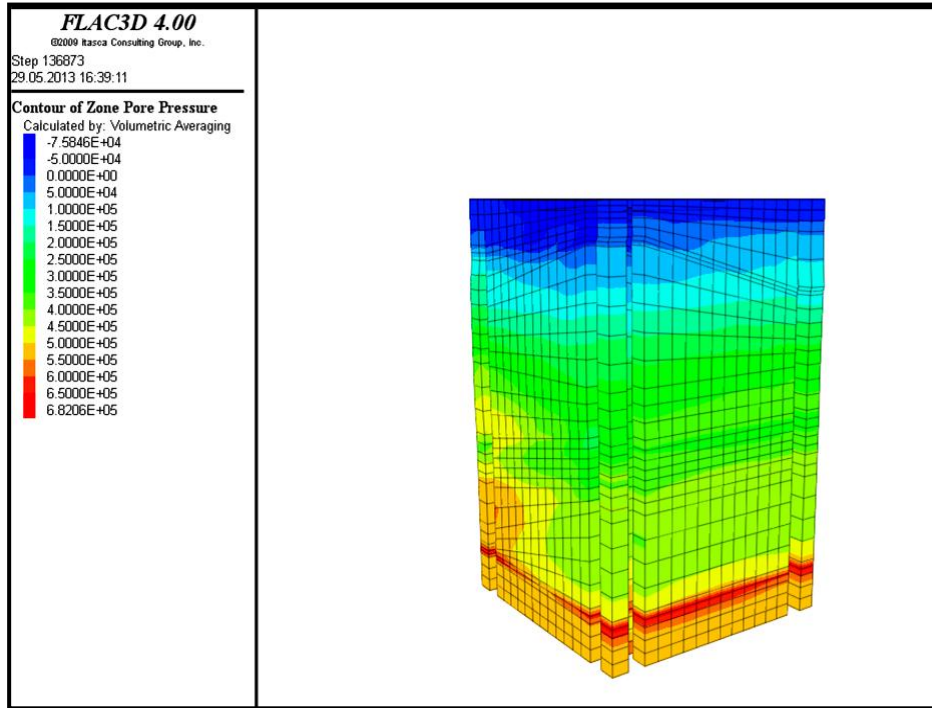


Figure 5.2 : The pore pressure values changes in layers in zone 1 for FDM dynamic analysis.

The chosen points for getting histories from model are given in Figure 5.3. Point 1, 2, 3 and 4 are representing the borehole properties and the point 5 is for a forecasting point for 3D FDM models which is in the middle of the zones.

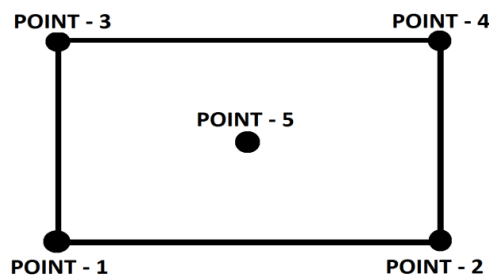


Figure 5.3 : Chosen points at zones for FDM 3D models.

The pore pressure ratio calculation is done by using (5.1).

$$Ru = \text{Pore Pressure} / \text{Total Stress} \text{ or}$$

$$Ru = U / \sigma_T \quad (5.1)$$

A pore pressure ratio – step graph for zone 1, point 3, layer 3 which is a silty sand is given in Figure 5.4.

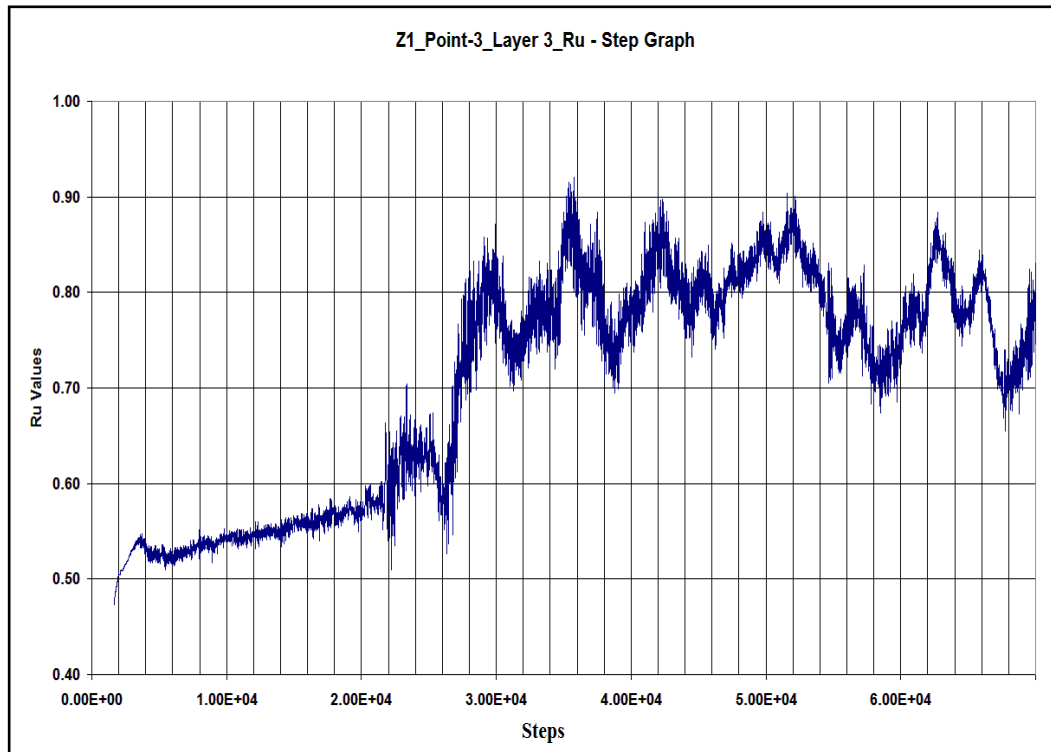


Figure 5.4 : Pore pressure ratio - step graph for zone 1, point 3, layer 3 as a silty sand layer.

Model success ratios are calculated by finding the difference between the expected static values that are calculated by simplified equations and the values of FDM results. The static maximum pore pressure values are calculated for the bottom layer of soils. The average success rate of models is calculated 95% and all zones' results are given in Table 5.1.

Table 5.1 : Success rates of models for all zones.

	Calculated Pore Pressure (kPa)	Model Pore Pressure (kPa)	Difference (kPa)	Success Rate (%)
Zone 1	540	511	29	0.95
Zone 2	540	521	19	0.97
Zone 3	441	414	27	0.94
Zone 4	441	421	20	0.95
Zone 5	441	425	16	0.96
Zone 6	441	418	23	0.95
Zone 7	422	403	19	0.96
Zone 8	392	368	24	0.94
Zone 9	402	386	16	0.96
Zone 10	402	371	31	0.92
Zone 11	412	380	32	0.92

Liquefaction potential for zone 1 is given in Figure 5.5 as a pore pressure ratio - depth graph with the results of developed 3D FDM based model for Zone 1. Pore pressure ratio (r_u) values which are equal to 1 means that that layer liquefy and about 0.8 to 1.0 means that layer has a potential to liquefy. The r_u values bigger than 1.0 are because of FLAC3D software calculation due to the complicated nature of problems.

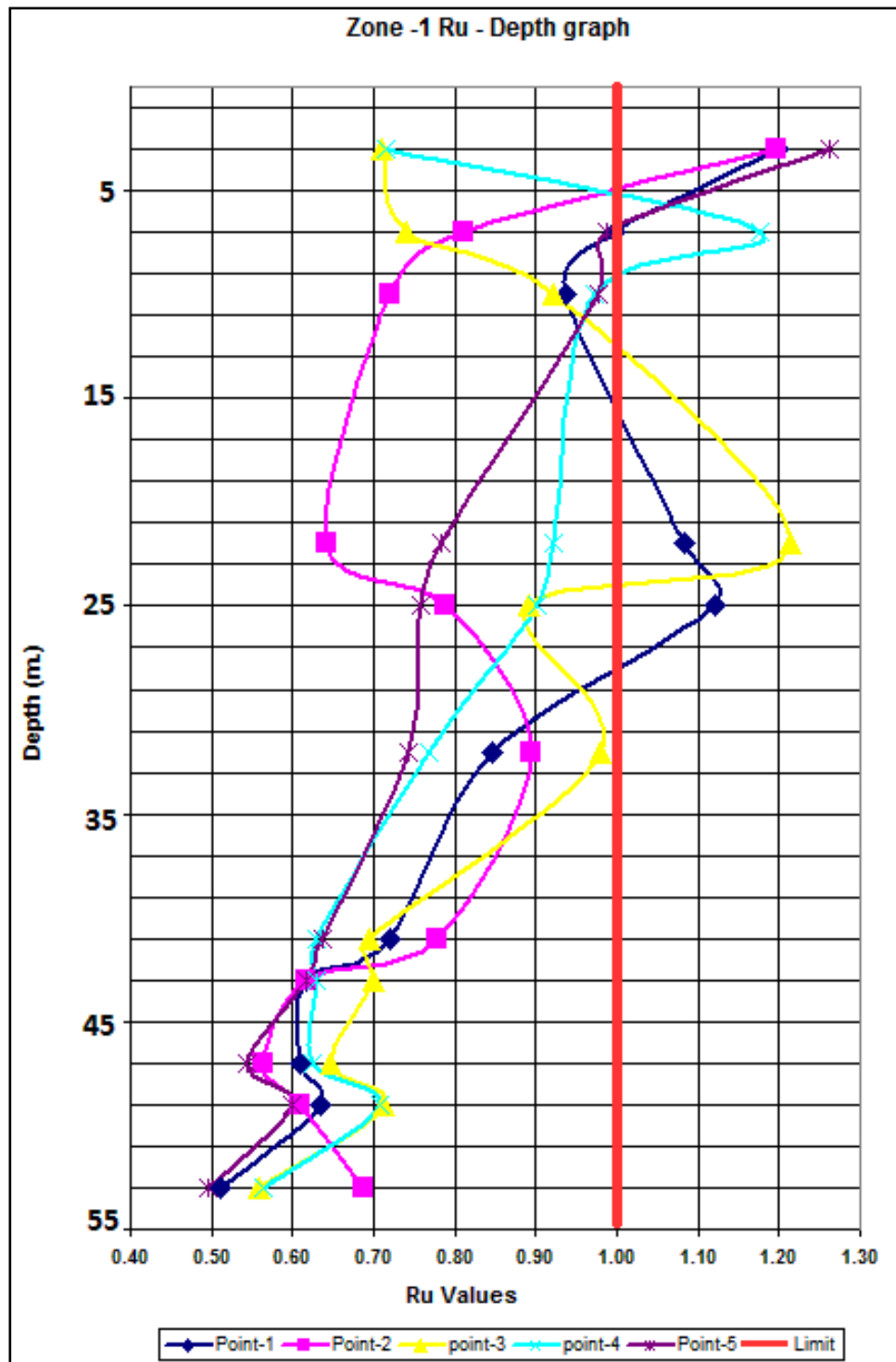


Figure 5.5 : Pore pressure ratio - depth graph for zone 1.

5.3 FDM Model Results of 1D – 2D and 3D Geometries for Zone 1 and Zone 8

Liquefaction potential analysis are performed by modeling the zones first of all in one dimensional by using the SPT log data and then continued on two dimensional geometry models by using the chosen SPT point with the neighbor SPT point. After getting the results from 1D and 2D models, the results are compared with the modeled 3D liquefaction analysis results. Liquefaction potential analysis for zone 1 is given in pore pressure ratio - depth graph by comparing the developed 1D, 2D and 3D model results in Figure 5.6.

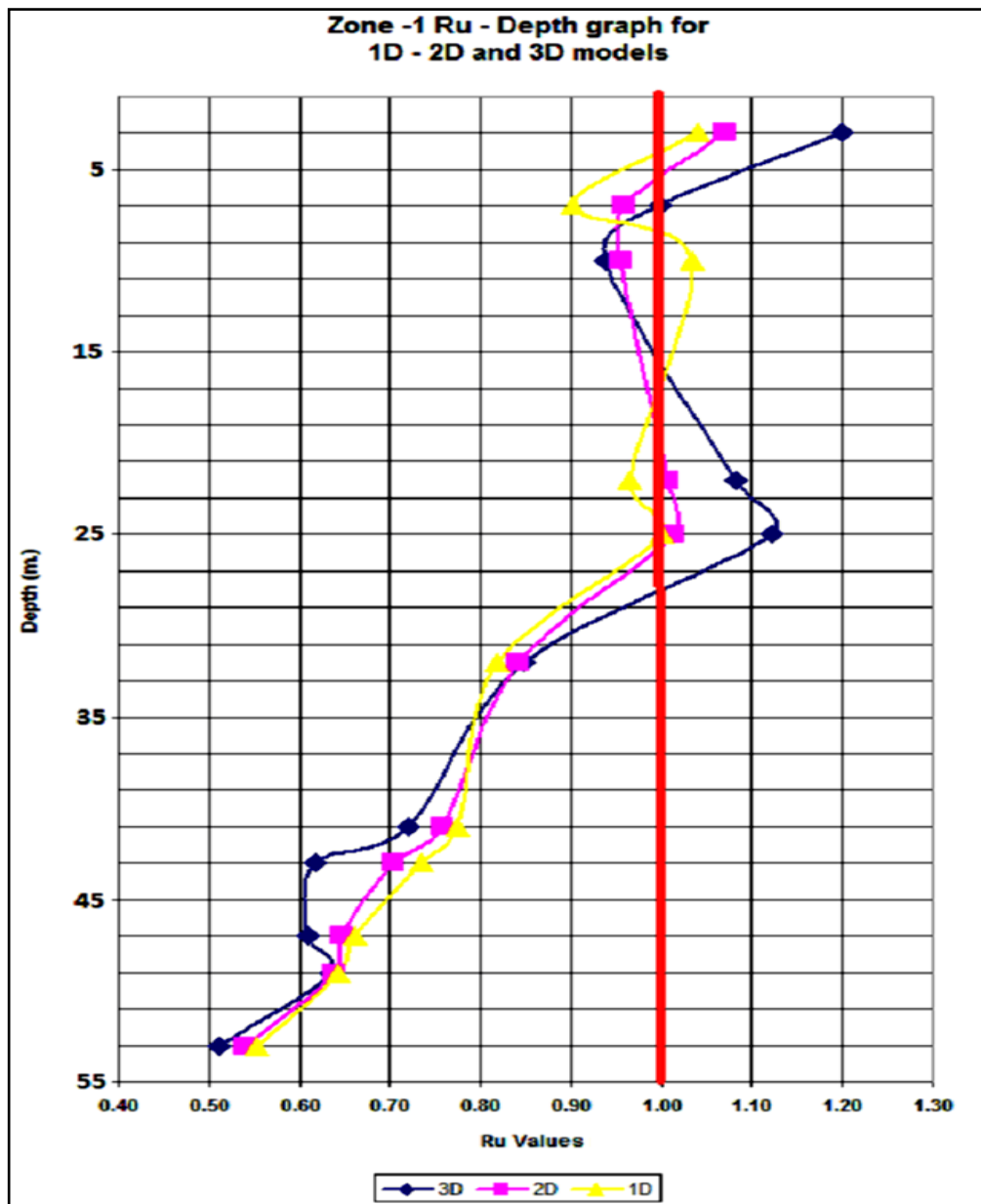


Figure 5.6 : Pore pressure ratio - depth graph in zone 1 for 1D – 2D and 3D FDM model.

Figure 5.7 is presenting the same comparison graph that is sharing the results of geometry differences comparison for liquefaction analysis based on FDM for zone 8.

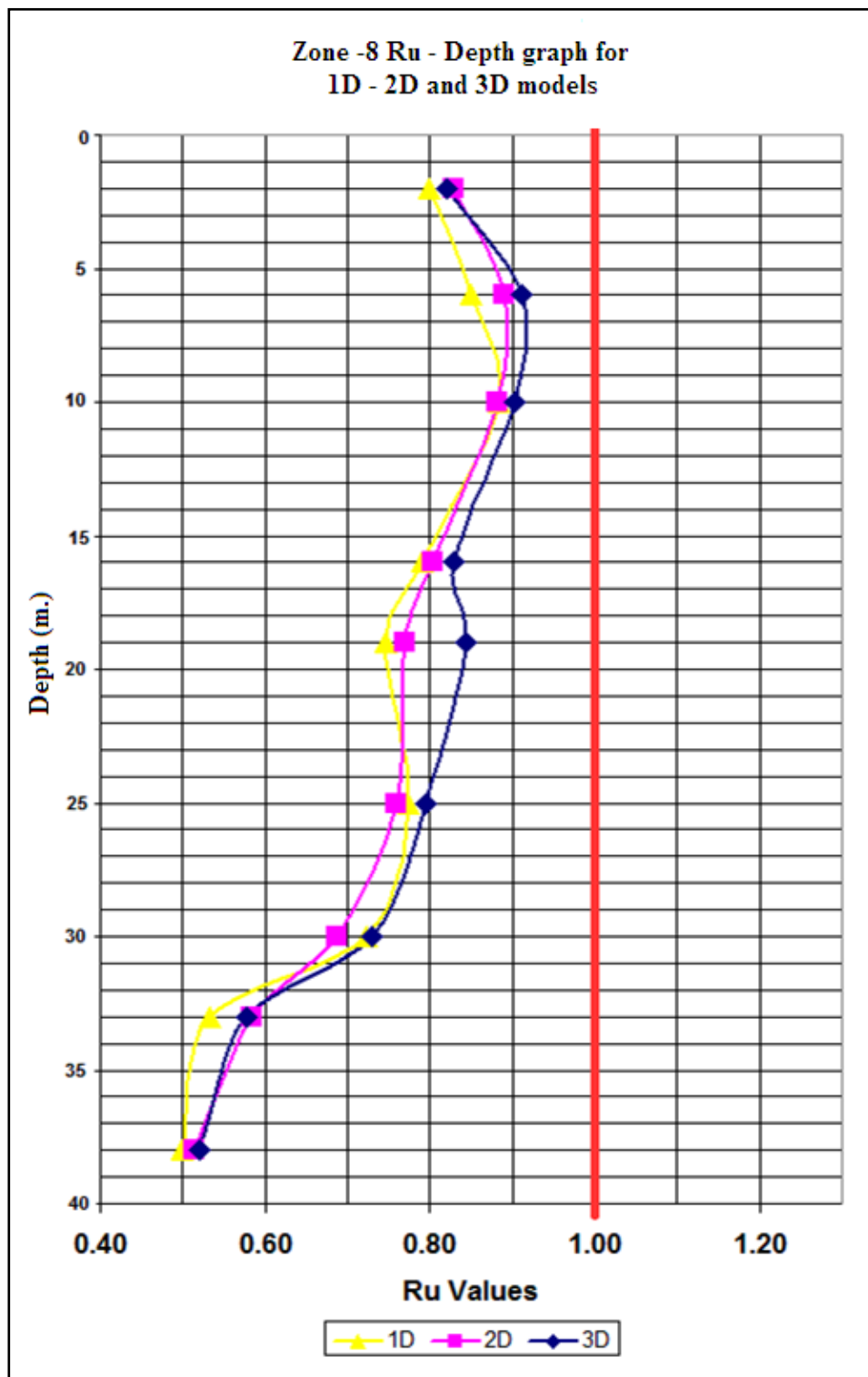


Figure 5.7 : Pore pore pressure ratio - depth graph in Zone 8 for 1D – 2D and FDM model in 3D.

5.4 Kurbağlıdere Region NN based Model Results

5.4.1. BPNN Result

The forecasted r_u values for zone 10 and zone 11 by using back propagation neural network model is given in Table 5.2. And also, in the table by using the FDM based results the differences (error values) and the error percentages are calculated for all layers at zone 10 and 11.

Table 5.2 : Forecasted r_u values by using BPNN.

NN	FDM	Error Number	Error Percentage
0.76	0.79	0.03	0.04
1.01	0.82	0.19	0.23
0.80	0.80	0.00	0.00
0.75	1.04	0.29	0.28
0.84	0.76	0.08	0.11
0.63	0.71	0.08	0.11
0.57	0.55	0.02	0.04
0.89	0.55	0.34	0.62
1.01	0.94	0.08	0.08
0.76	0.78	0.02	0.03
0.75	0.88	0.13	0.15
0.84	0.61	0.23	0.38
0.62	0.58	0.04	0.08
0.57	0.53	0.04	0.07
0.77	0.50	0.26	0.52
1.01	0.96	0.04	0.05
0.86	0.80	0.06	0.08
0.75	0.86	0.11	0.13
0.71	0.75	0.03	0.05
0.67	0.58	0.08	0.14
0.57	0.41	0.16	0.40
0.76	0.86	0.11	0.12
1.02	0.72	0.30	0.41
0.75	0.86	0.11	0.13
0.75	1.01	0.26	0.26
0.71	0.82	0.11	0.13
0.65	0.68	0.04	0.05
0.57	0.39	0.18	0.45
0.76	0.92	0.16	0.18
1.01	0.96	0.04	0.04
0.78	0.82	0.05	0.06
0.75	0.86	0.11	0.12
0.92	0.72	0.21	0.29
0.63	0.44	0.19	0.43
0.57	0.42	0.15	0.35
1.01	0.49	0.51	1.04
0.76	0.88	0.12	0.14

Table 5.2 (continued): Forecasted r_u values by using BPNN.

N.N.	Flac 3D	Error Number	Error Percentage
0.78	0.85	0.07	0.09
0.97	0.74	0.23	0.31
0.62	0.70	0.07	0.11
0.63	0.69	0.06	0.09
0.51	0.44	0.06	0.14
1.00	0.57	0.44	0.77
0.73	0.87	0.14	0.16
0.78	0.74	0.04	0.06
0.91	0.84	0.07	0.09
0.62	0.77	0.14	0.18
0.63	0.61	0.01	0.02
0.51	0.47	0.03	0.07
0.87	0.80	0.07	0.09
0.75	0.93	0.18	0.19
0.75	0.79	0.04	0.05
0.90	0.86	0.03	0.04
0.65	0.75	0.10	0.13
0.66	0.70	0.04	0.06
0.51	0.48	0.02	0.05
1.00	0.68	0.32	0.47
0.73	0.95	0.22	0.24
0.78	0.95	0.18	0.19
0.91	0.94	0.02	0.03
0.62	0.77	0.15	0.20
0.63	0.70	0.08	0.11
0.51	0.47	0.03	0.07
1.01	0.87	0.13	0.15
0.73	0.94	0.22	0.23
0.79	0.82	0.04	0.04
0.97	0.88	0.09	0.10
0.62	0.62	0.00	0.00
0.63	0.48	0.15	0.32
0.53	0.40	0.13	0.32

The success of calibration is presented in the error – epoch's elapsed graph in Figure 5.8 and as it is seen the errors are approaching to zero by the epochs continue which means the model is succeeded. The success rate of BPNN model is evaluated by calculating error percentages for all layers as seen in Table 5.3.

Table 5.3 : Success rate of BPNN model.

Average Error Number	0.12
Error Percentage	0.18
Success Rate of Model	0.82

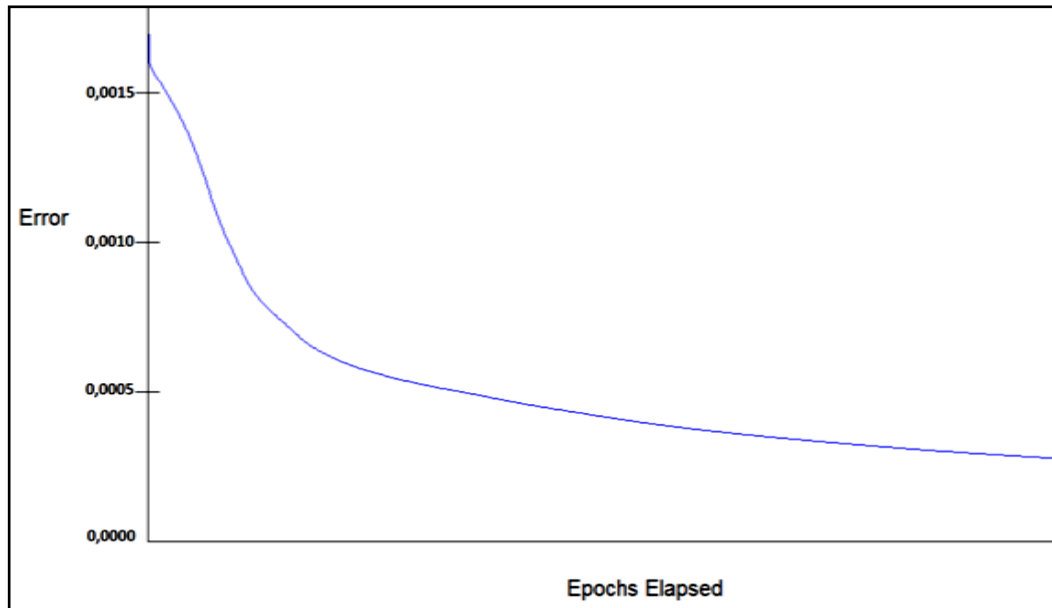


Figure 5.8 : Error – epochs elapsed graph of BPNN model.

The contribution factors for BPNN Model that means which parameter effects the liquefaction potentials for site is given in the Table 5.4 and also in Figure 5.9.

Table 5.4 : The contribution factors for BPNN Model.

Friction Angle (degree)	0.19826
SPT - N_{60}	0.15296
Density (kg/m^3)	0.13701
Water Table (m)	0.08231
Thickness of Layer (m)	0.06826
Cohesion (Pa)	0.03928
Bulk Modulus (Pa)	0.03662
Shear Modulus (Pa)	0.0342

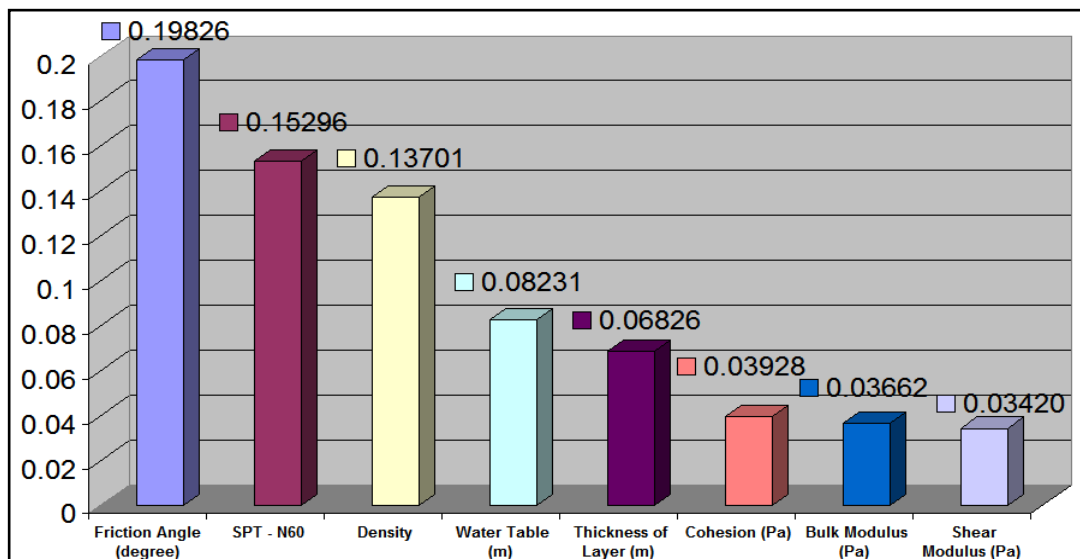


Figure 5.9 : The contribution factors (parameters) graph for BPNN Model.

Comparisons of FDM model and NN model based on BPNN results for Zone 10 (Figure 5.10) and Zone 11 (Figure 5.11) at middle of area (point 5) are given below.

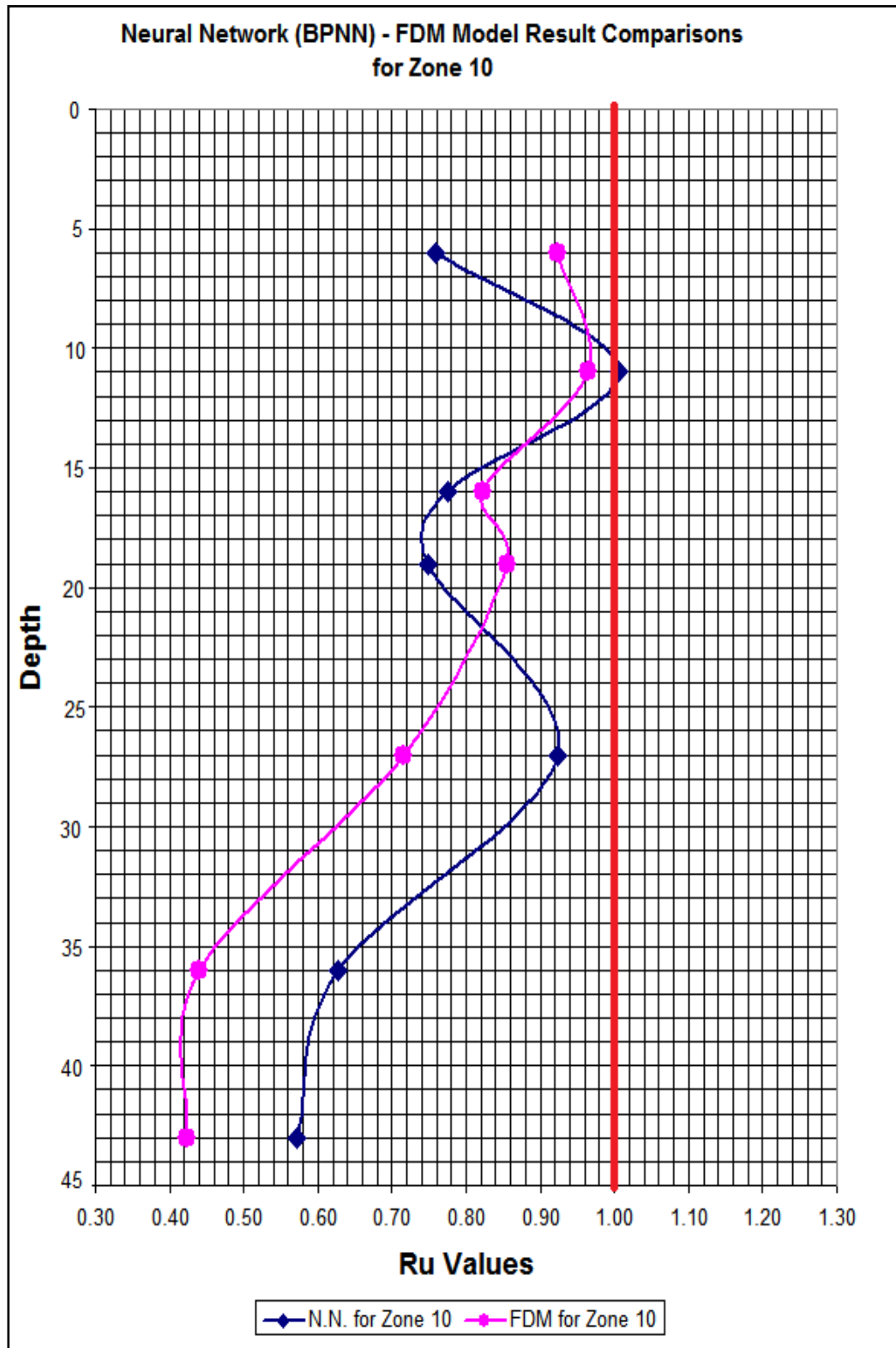


Figure 5.10 : BPNN and FDM model results comparisons for zone 10.

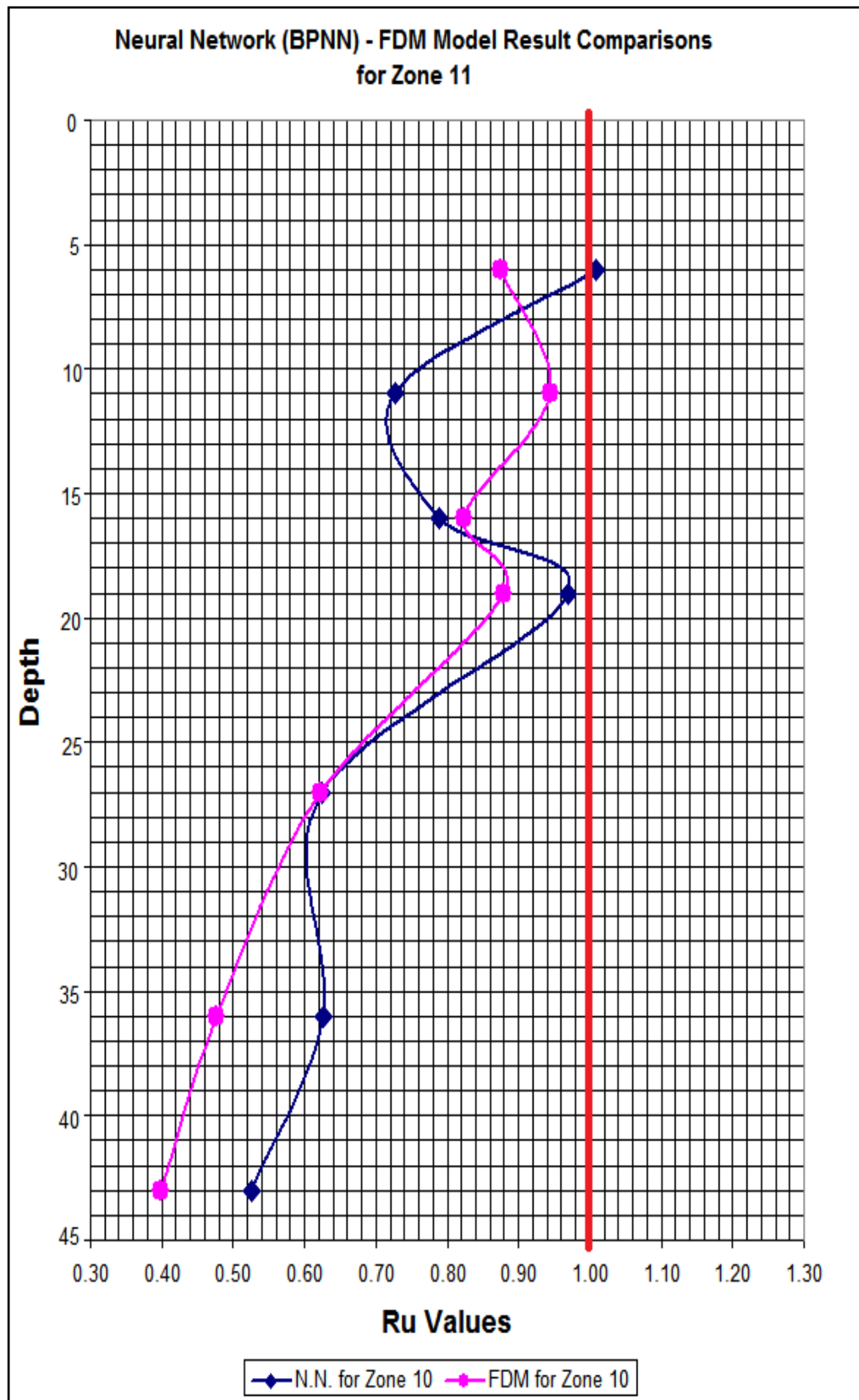


Figure 5.11 : BPNN and FDM model results comparisons for zone 11.

5.4.2. GRNN Result

The forecasted r_u values for zone 10 and zone 11 by using general regression neural network model is given in Table 5.5.

Table 5.5 : Forecasted r_u values by using GRNN.

N.N.	Flac 3D	Error Number	Error Percentage
0.78	0.79	0.02	0.02
1.04	0.82	0.22	0.27
0.81	0.80	0.01	0.01
0.72	1.04	0.32	0.31
0.89	0.76	0.13	0.18
0.66	0.71	0.05	0.07
0.51	0.55	0.04	0.08
0.78	0.55	0.23	0.42
1.04	0.94	0.11	0.11
0.81	0.78	0.03	0.04
0.72	0.88	0.16	0.18
0.89	0.61	0.29	0.47
0.66	0.58	0.08	0.14
0.59	0.53	0.06	0.10
0.78	0.50	0.27	0.54
1.04	0.96	0.08	0.08
0.81	0.80	0.01	0.01
0.72	0.86	0.14	0.17
0.89	0.75	0.15	0.20
0.66	0.58	0.08	0.13
0.47	0.41	0.06	0.14
0.78	0.86	0.09	0.10
1.04	0.72	0.32	0.45
0.81	0.86	0.05	0.06
0.72	1.01	0.29	0.29
0.89	0.82	0.07	0.09
0.66	0.68	0.03	0.04
0.44	0.39	0.04	0.10
0.78	0.92	0.15	0.16
1.04	0.96	0.08	0.08
0.81	0.82	0.01	0.01
0.72	0.86	0.14	0.16
0.89	0.72	0.18	0.25
0.66	0.44	0.22	0.49
0.50	0.42	0.07	0.17
0.77	0.49	0.28	0.56
0.96	0.88	0.08	0.09
0.94	0.85	0.08	0.10
1.03	0.74	0.30	0.40

Table 5.5 (continued): Forecasted r_u values by using GRNN.

N.N.	Flac 3D	Error Number	Error Percentage
0.70	0.70	0.01	0.01
0.66	0.69	0.03	0.05
0.51	0.44	0.06	0.14
0.96	0.57	0.39	0.69
0.96	0.87	0.09	0.11
0.94	0.74	0.20	0.27
1.03	0.84	0.19	0.23
0.70	0.77	0.06	0.08
0.66	0.61	0.04	0.07
0.57	0.47	0.09	0.20
0.96	0.80	0.16	0.20
0.96	0.93	0.03	0.03
0.94	0.79	0.15	0.18
1.03	0.86	0.17	0.20
0.70	0.75	0.05	0.06
0.66	0.70	0.04	0.06
0.54	0.48	0.06	0.12
0.96	0.68	0.28	0.40
0.96	0.95	0.01	0.01
0.94	0.95	0.02	0.02
1.03	0.94	0.10	0.10
0.70	0.77	0.07	0.09
0.66	0.70	0.05	0.07
0.57	0.47	0.10	0.21
0.96	0.87	0.08	0.10
0.96	0.94	0.02	0.02
0.94	0.82	0.11	0.14
1.03	0.88	0.15	0.17
0.70	0.62	0.08	0.13
0.66	0.48	0.18	0.38
0.47	0.40	0.07	0.19

The success rate of GRNN model is evaluated by calculating error percentages for all layers as seen in Table 5.6. The model has an 83% success rate.

Table 5.6 : Success rate of GRNN model.

Average Error Number	0.11
Error Percentage	0.17
Success Rate of Model	0.83

The success of calibration is presented in the error – epochs elapsed graph and is given in Figure 5.12.

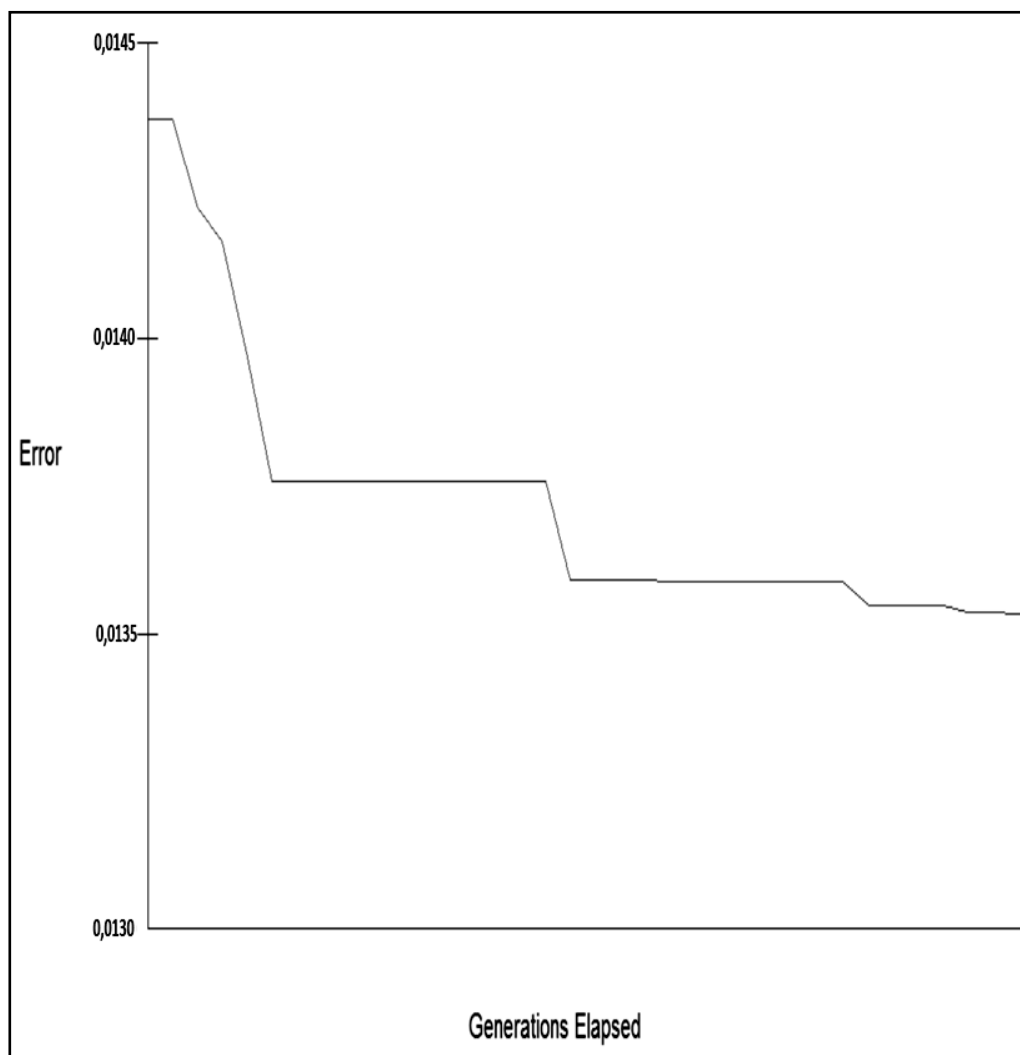


Figure 5.12 : Error – epochs elapsed graph of GRNN model.

The comparisons of FDM model with NN model based on GRNN results are given in Figure 5.13 and Figure 5.14.

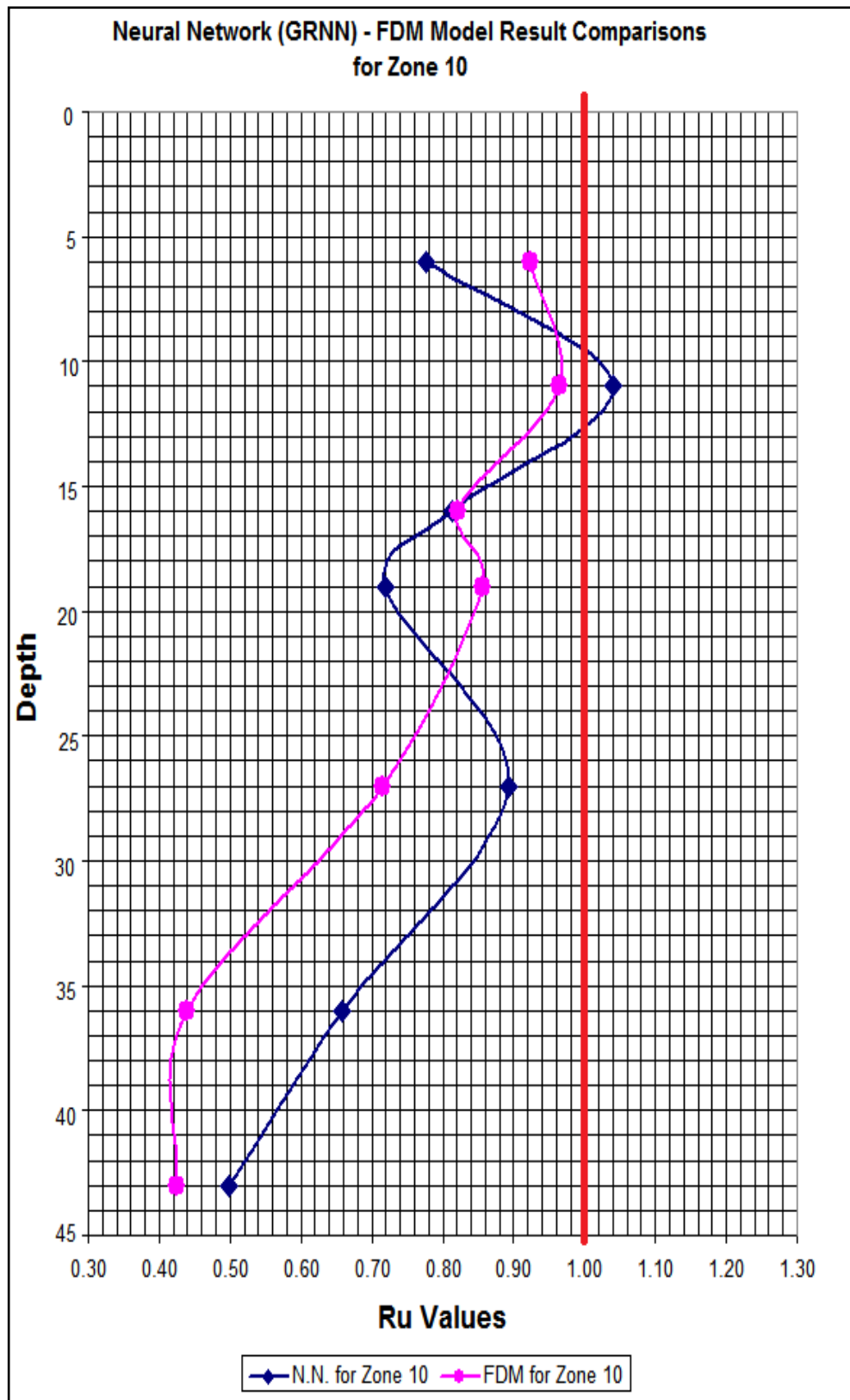


Figure 5.13 : GRNN and FDM model results comparisons for zone 10.

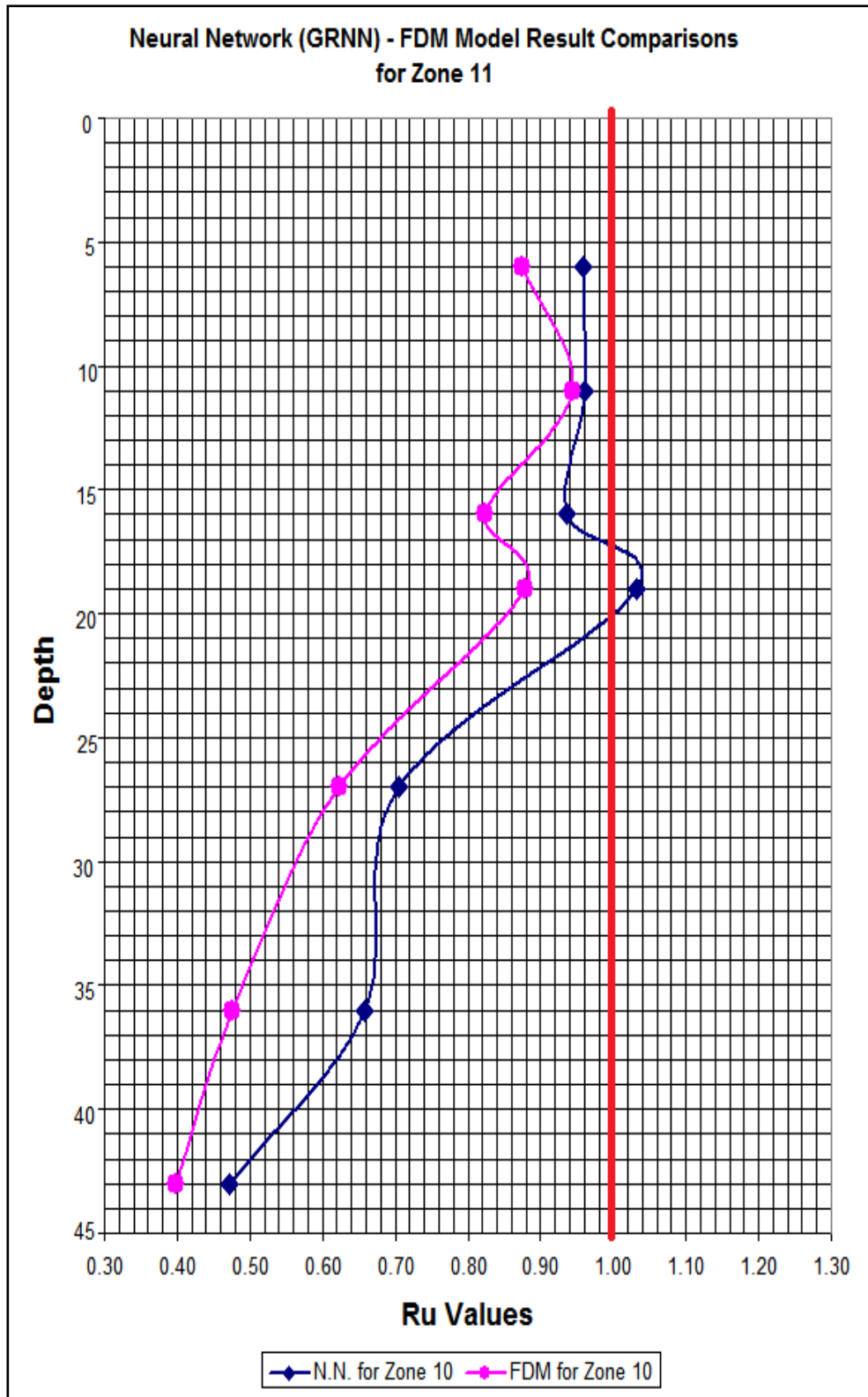


Figure 5.14 : GRNN and FDM model results comparisons for zone 11.

A general comparison done for Kurbağlıdere region at zone 10 and zone 11 which are the forecasted areas for liquefaction occurrence by comparing the result both FDM in 3D, GRNN and BPNN is given in Figure 5.15 and 5.16.

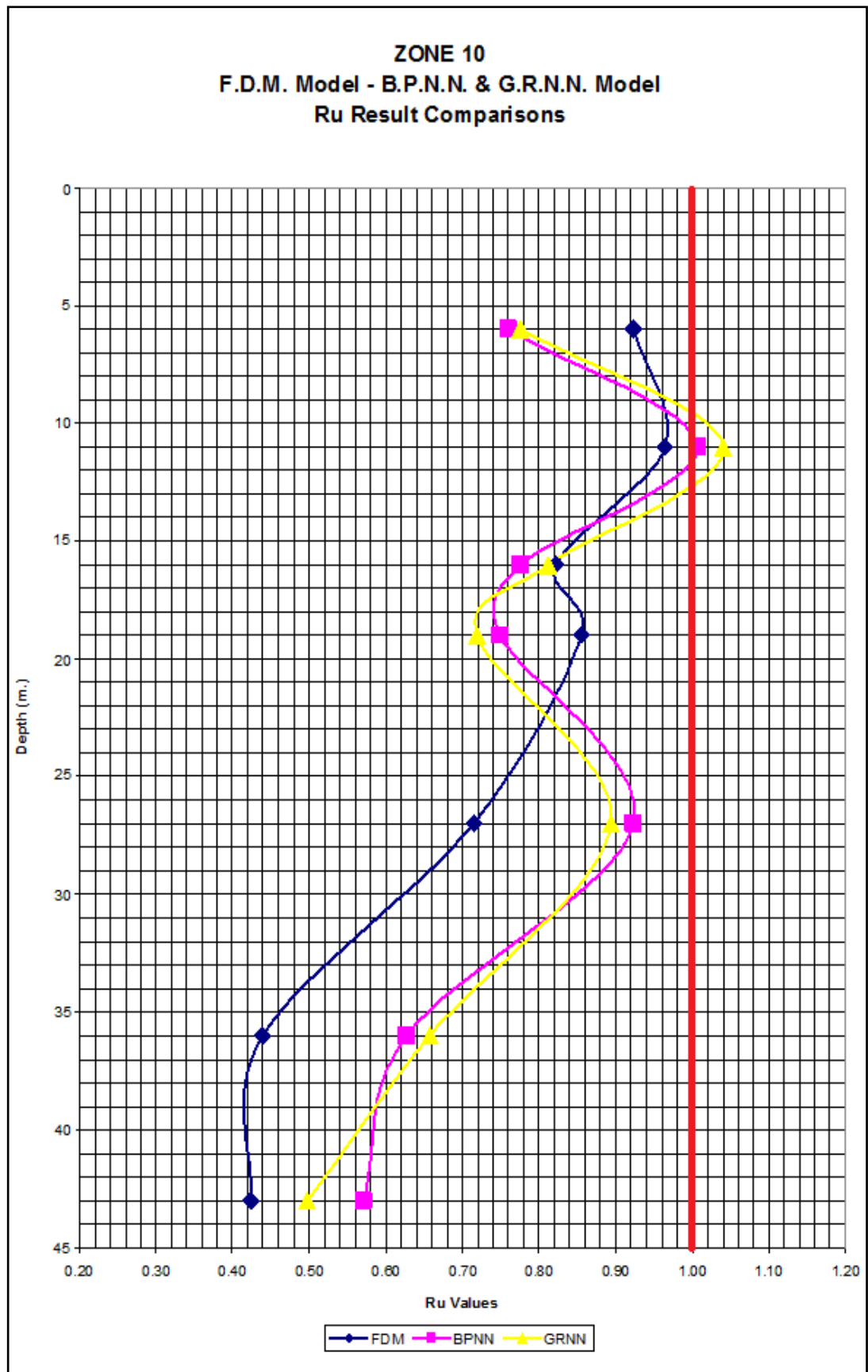


Figure 5.15 : GRNN, BPNN and FDM in 3D model results comparisons for zone 10.

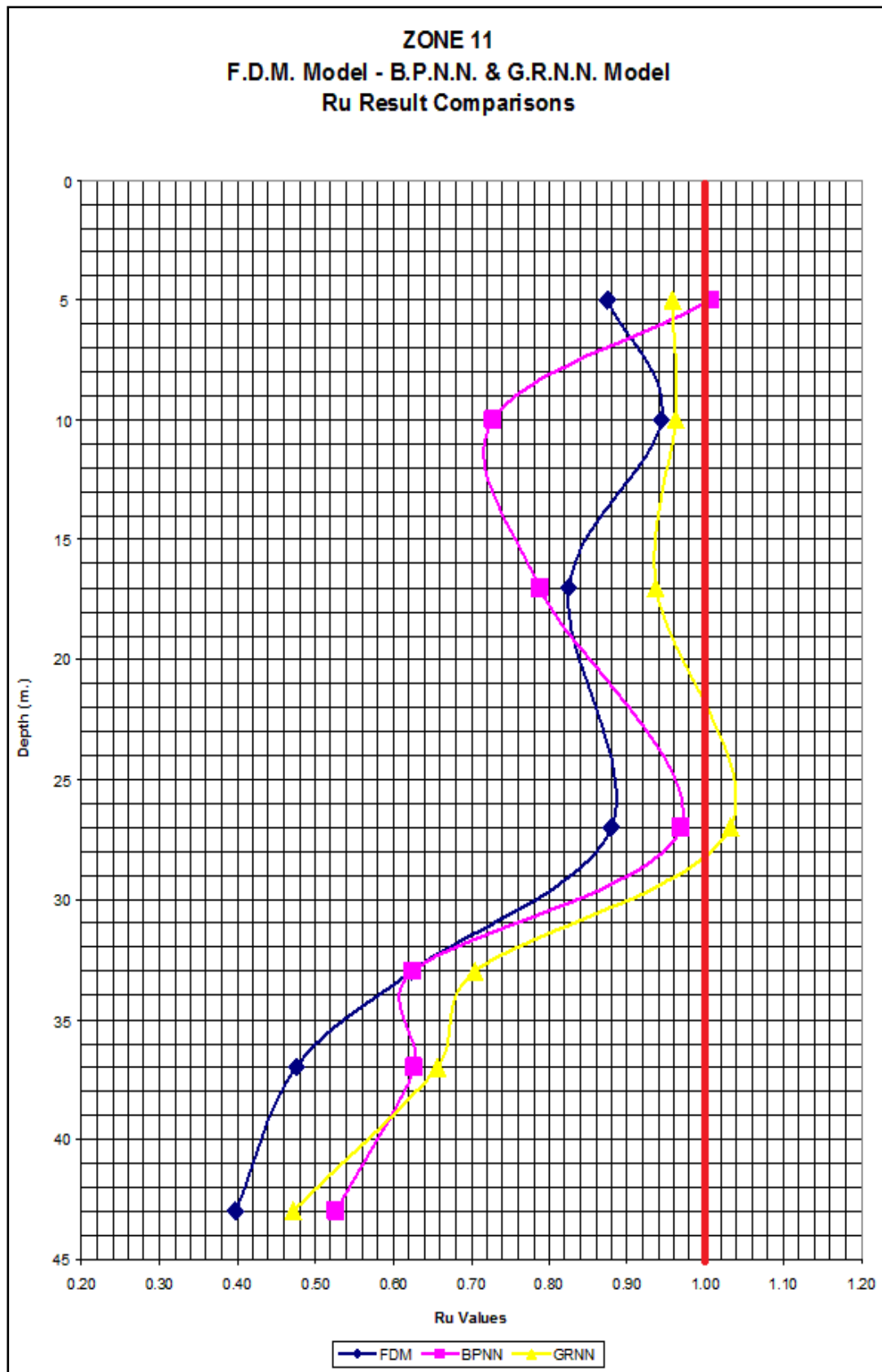


Figure 5.16 : GRNN, BPNN and FDM in 3D model results comparisons for zone 11.

6. CONCLUSION AND RECOMMENDATIONS

6.1 Summary

In this study, the aim is to compare different methods used in evaluating liquefaction potential and their results and also to determine the parameters effects in a sensitivity study. To determine the liquefaction occurrence effects on soils by using pore pressure ratios, the finite difference method based models considered a comparison of one dimensional, two dimensional and three dimensional models.

After that the results are also compared with the developed neural network model results and the general results. As a result of this study, scientists can attain knowledge and forecast liquefaction potential of a chosen case study area.

6.2 Conclusion

In conclusion, an extended literature research on liquefaction phenomena and various liquefaction analysis methods are investigated. A literature research on current and cutting-edge computer tools to evaluate liquefaction potential of soils is presented in detail. Case study selected for thesis is from Kurbağlıdere, Istanbul which is known to have a likelihood of liquefaction potential. Soil data are taken from Istanbul microzonation project, carried out by Istanbul Metropolitan Municipality.

Modeling the soil in 3D and with artificial intelligence tools had challenges. There were difficulties in the first developed model, which is the finite difference method based model, due to modeling the layered nature of the soil. It is seen that for soil that has up to three to four layers, the developed models can evaluate pore pressures easier than soils with ten to thirteen layered profiles. Another complexity in FDM models is seen in modeling large scale areas (ie., 250 m. x 250 m.) requires lengthy computer run times. Therefore, while modeling large areas, the decision of using a scaling has emerged, with acceptable run times. If it is preferred to use the 3D FDM models for microzonation project areas larger than 100 m × 100 m, a scaling procedure for mesh sizes must be used.

It is seen that the run time for 3D FDM models is over 22 hours if the mesh size reaches 250 m². On the other hand, if the mesh size is reduced to 25m² by using a scale number, the run time is 8 hours on average. Utilizing an optimum damping type and number changes the run time of the models. Layered soil profiles with different and varying numbers of layers also lead to a significant change in run times. On the other hand, while creating the FDM models, the earthquake motion must be accurately defined; otherwise the results will not represent the true behavior in the field.

It can be stated that liquefaction analysis performed with FLAC3D software are more suitable for liquefaction analysis under or near structures, therefore, analysis of soil - structure interaction problems can be solved correctly.

The 3D model also had interesting results for silty soil. In 1985, Ishihara stated that clay or silt materials having a low plasticity index value will exhibit physical characteristics resembling those of cohesionless soils, and thus have a high degree of potential for liquefaction. Also, Walker and Steward in 1989 stated that non-plastic and low plasticity silts, despite having their grain size distribution curves outside of Tsuchida's boundaries for soils susceptible to liquefaction, do have a potential for liquefaction, similar to that of sands. Based on this knowledge, in the three-dimensional FDM models, some subzones of the study area of clay or silt show a potential for liquefaction. However, in one or two-dimensional FDM models, this geotechnical phenomenon cannot be observed.

As cyclic mobility can occur in both loose and dense soils in a subzone of the study area, after the depth of 20 meters, liquefaction probability has been seen. This result shows that cyclic mobility occurrence can be observed if one uses 3D FDM models. These two results show the power of three-dimensional modeling in FDM.

In addition, it is seen that the developed FDM model is a useful model for forecasting the liquefaction potentials of a point in the case study area which one might not have complete soil information. The comparison of simplified method results and the FDM model results are close and meaningful and the success rates of the developed models have an average success rate of 92%.

In conclusion of the FDM results, the developed models are useful and simple for 2D models and recommended for first impressions of the area about liquefaction potentials. Also, it is useful but complicated to model in 3D because of the challenge in creating appropriate geometry, defining soil properties and lengthy model run times. Therefore, this method is recommended for examining soils especially under structures, not for vast microzonation projects.

In the neural network based models, it is seen that the friction angle of soils and the SPT-N corrected values are the most effective parameters for liquefaction potentials of soils. The success rates of the developed NN models are meaningful, with an average success rate of 82%. It can be said that NN models are useful for forecasting studies for liquefaction analysis. If the database can be fed with more datasets from different case study areas with different earthquakes, the model success rate is expected to increase.

In conclusion, with the comparison of FDM model results and NN based model results, the maximum approximate value difference of 0.2 for the calculated pore pressure ratios is seen. This study presents a new approach to liquefaction problems of soils during earthquakes. In this study, the reliability of results and the error limits of developed models are succeeded, therefore this study brings a new view point to earthquake engineering and geotechnical engineering and will assist in future research and case studies.

6.3 Recommendations for Future Works

In the future, a forecasting for different case study areas can be performed by using these two different models. Also, a more advanced neural network model can be created if the NN model is fed with more datasets. Therefore, by solving and forecasting possible soil engineering problems, liquefaction analysis are simplified thru by working on more 3D models with different soil types and possible earthquake motions. For future works, an additional search can be done for defining different earthquake motions in both of the models to compare and understand the pore pressure changes in layered soils and to understand the behavior of defined Finn model calculation steps in the developed FDM based models.

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APPENDICES

APPENDIX A : Sample SPT soil profile used for zone 1.

APPENDIX B : Calculated and defined soil property parameters for FDM models.

APPENDIX C : Finn constants calculated for all zones in FDM models.

APPENDIX D : Soil property groups for zone 1 to zone 11.

APPENDIX E : Evaluated pore pressure ratio's for all zones.

APPENDIX F : Evaluated pore pressure ratio – depth graphs for all zones.

APPENDIX G : Input parameters for neural network models.

APPENDIX A : Sample SPT soil profile used for zone 1.

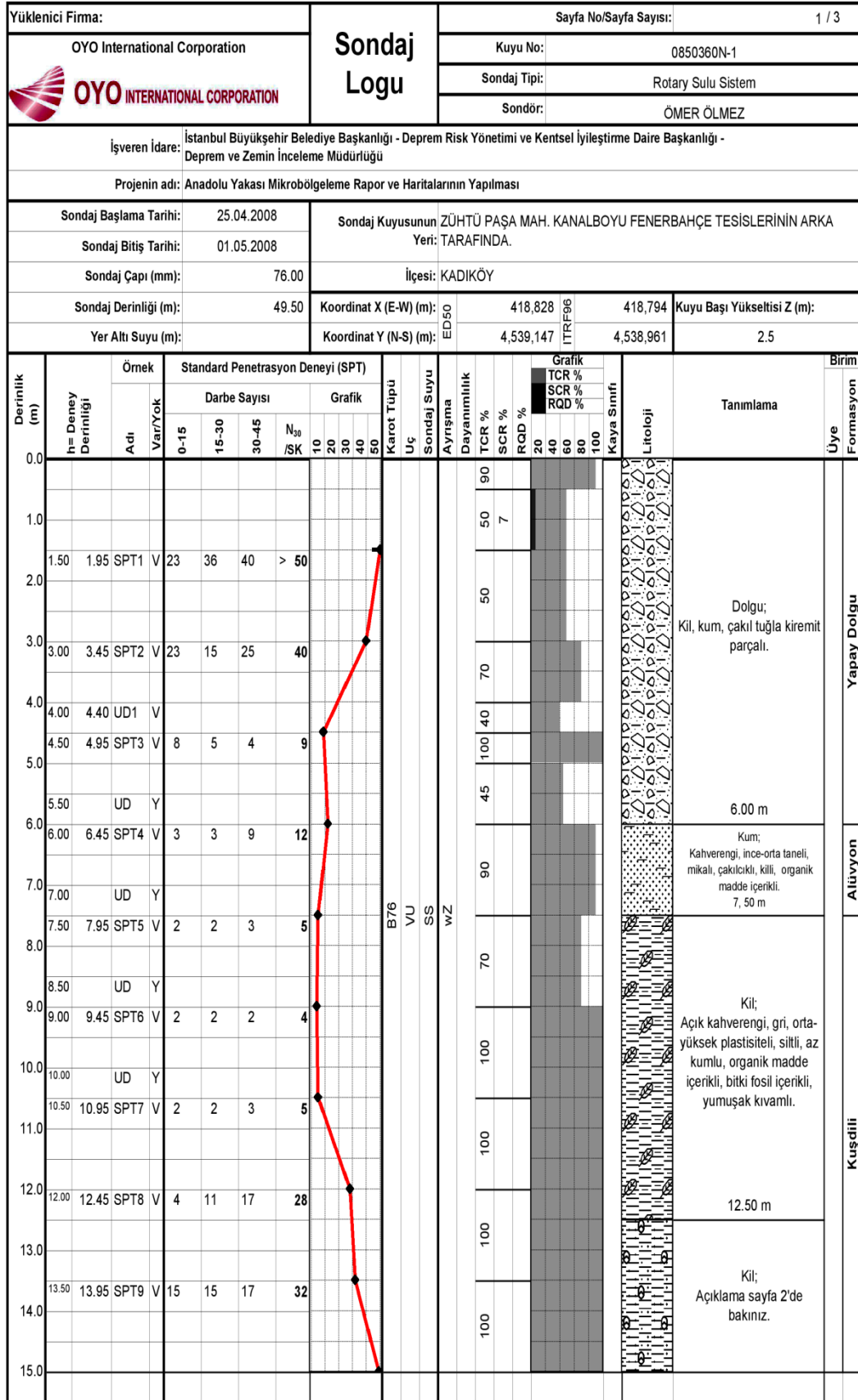


Figure A.1 : Sample SPT soil profile used for zone 1.

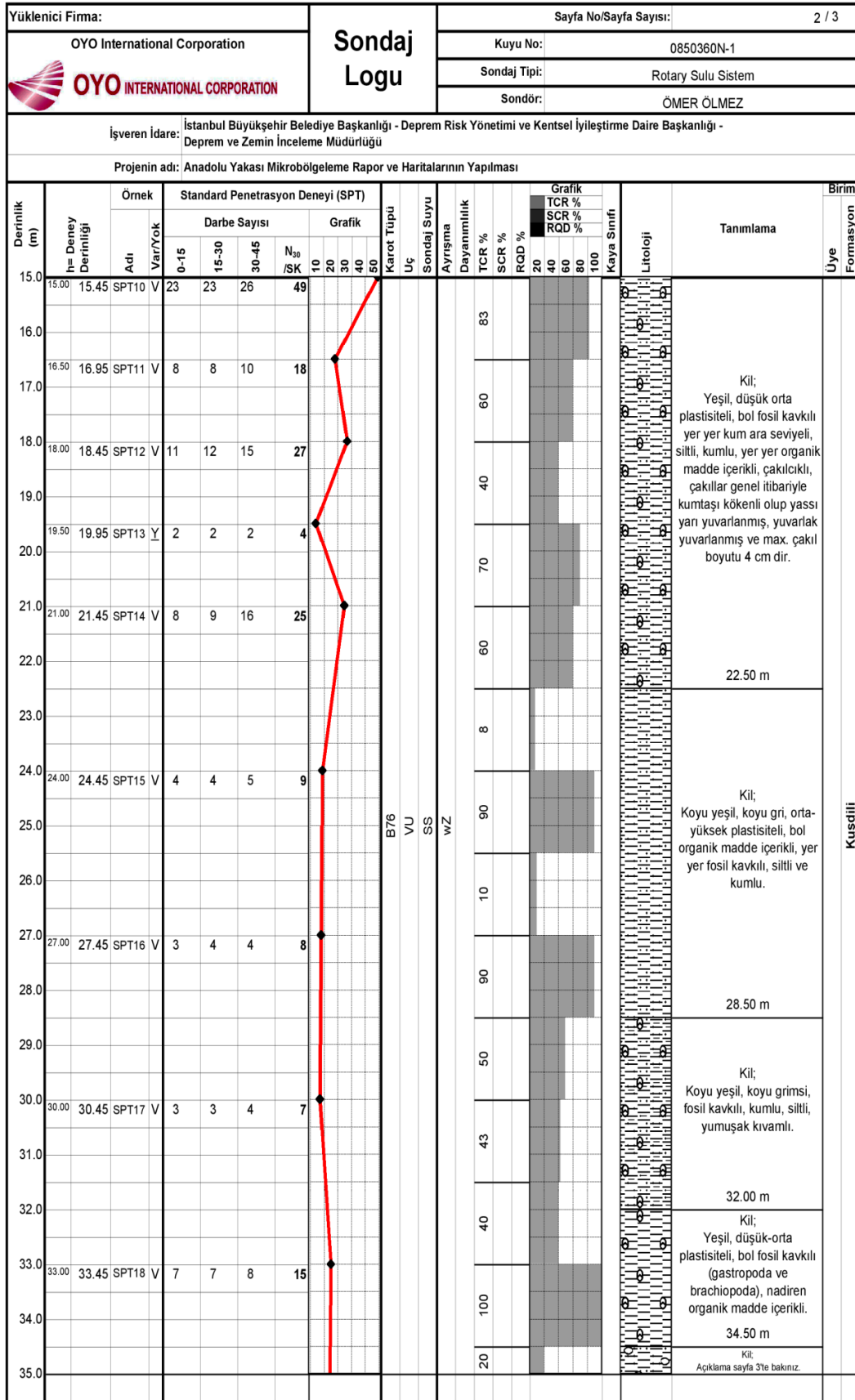


Figure A.1 (continued): Sample SPT soil profile used for zone 1.

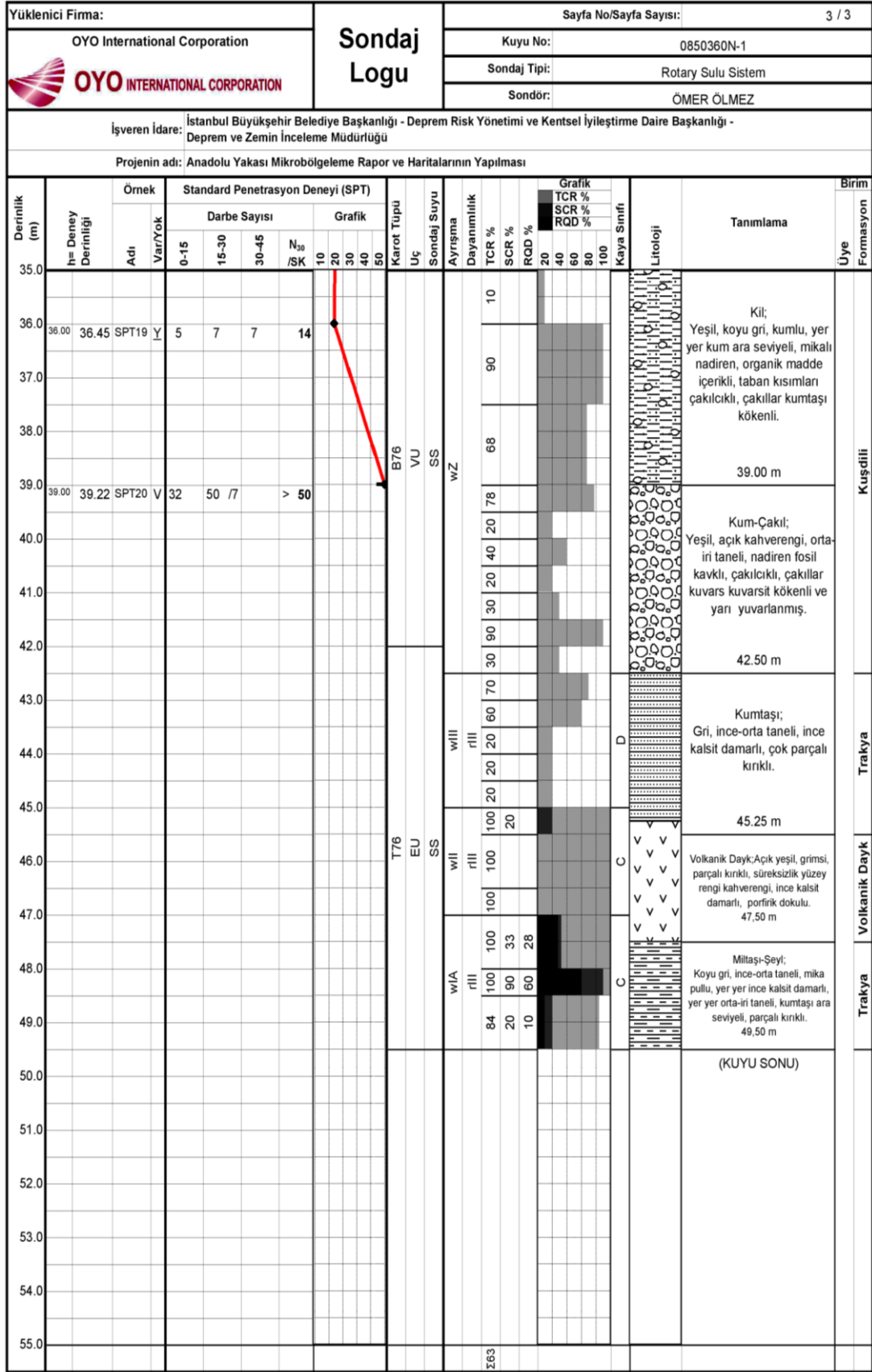


Figure A.1 (continued): Sample SPT soil profile used for zone 1.

APPENDIX B : Calculated and defined soil property parameters for FDM models.

Table B.1 : Calculated and defined soil properties for FDM models.

		SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)
ZONE 1	ArtificialFill-1	37	6	3.0E+06	5.0E+06	1700	20	2.0E+04
	Sand-1b	8	2	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltySand-1	15	4	1.5E+07	3.3E+07	1950	30	0.0E+00
	ClayeySand-1	10	10	4.0E+07	8.0E+07	1850	28	2.0E+04
	SandyClay-1	2	6	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay-1	3	11	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand-1a	18	3	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltyClay-1	18	3	2.0E+07	4.0E+07	2000	25	2.0E+04
	Claystone-1	17	3	4.0E+07	8.0E+07	2200	35	5.0E+04
	Sandstone-1	17	2	8.0E+07	1.6E+08	2200	40	0.0E+00
	Rock-1	24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill-1	37	2	3.0E+06	5.0E+06	1700	20	2.0E+04
	Sand-1b	8	2	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltySand-1	15	1	1.5E+07	3.3E+07	1950	30	0.0E+00
	ClayeySand-1	10	16	4.0E+07	8.0E+07	1850	28	2.0E+04
	SandyClay-1	2	6	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay-1	3	6	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand-1a	18	3	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltyClay-1	18	6	2.0E+07	4.0E+07	2000	25	2.0E+04
	Claystone-1	17	6	4.0E+07	8.0E+07	2200	35	5.0E+04
	Sandstone-1	17	2	8.0E+07	1.6E+08	2200	40	0.0E+00
	Rock-1	24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill-1	37	8	3.0E+06	5.0E+06	1700	20	2.0E+04
	Sand-1b	8	2	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltySand-1	15	3	1.5E+07	3.3E+07	1950	30	0.0E+00
	ClayeySand-1	10	13	4.0E+07	8.0E+07	1850	28	2.0E+04
	SandyClay-1	2	6	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay-1	3	4	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand-1a	18	6	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltyClay-1	18	3	2.0E+07	4.0E+07	2000	25	2.0E+04
	Claystone-1	17	4	4.0E+07	8.0E+07	2200	35	5.0E+04
	Sandstone-1	17	1	8.0E+07	1.6E+08	2200	40	0.0E+00
	Rock-1	24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill-1	37	4	3.0E+06	5.0E+06	1700	20	2.0E+04
	Sand-1b	8	7	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltySand-1	15	1	1.5E+07	3.3E+07	1950	30	0.0E+00
	ClayeySand-1	10	9	4.0E+07	8.0E+07	1850	28	2.0E+04
	SandyClay-1	2	6	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay-1	3	6	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand-1a	18	3	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltyClay-1	18	6	2.0E+07	4.0E+07	2000	25	2.0E+04
	Claystone-1	17	6	4.0E+07	8.0E+07	2200	35	5.0E+04
	Sandstone-1	17	2	8.0E+07	1.6E+08	2200	40	0.0E+00

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

			SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 1	Rock-1	Point - 5	24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill-1		37	6	3.0E+06	5.0E+06	1700	20	2.0E+04
	Sand-1b		8	2	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltySand-1		15	4	1.5E+07	3.3E+07	1950	30	0.0E+00
	ClayeySand-1		10	10	4.0E+07	8.0E+07	1850	28	2.0E+04
	SandyClay-1		2	6	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay-1		3	11	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand-1a		18	3	5.0E+07	1.0E+08	1800	35	0.0E+00
	SiltyClay-1		18	3	2.0E+07	4.0E+07	2000	25	2.0E+04
	Claystone-1		17	3	4.0E+07	8.0E+07	2200	35	5.0E+04
	Sandstone-1		17	2	8.0E+07	1.6E+08	2200	40	0.0E+00
	Rock-1		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
ZONE 2	ArtificialFill	Point - 1	10	1	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		5	1	1.6E+07	3.5E+07	1850	26	1.0E+04
	Sand		4	1	1.5E+07	3.3E+07	1800	30	0.0E+00
	Sandstone		34	11	1.0E+08	2.0E+08	2000	40	1.0E+06
	Clay		13	1	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		16	6	5.0E+07	1.0E+08	1850	28	2.0E+04
	SandyClay		7	6	2.5E+07	5.0E+07	1850	30	1.0E+04
	Clay		9	6	2.0E+07	4.0E+07	1900	22	1.0E+04
	Sand		20	3	6.0E+07	1.2E+08	1800	36	0.0E+00
	SiltyClay		18	6	2.0E+07	4.0E+07	2000	25	2.0E+04
	Clay		18	3	3.5E+07	7.0E+07	1900	22	3.0E+04
	Sandstone		17	5	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 2	10	1	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		5	1	1.6E+07	3.5E+07	1850	26	1.0E+04
	Sand		4	3	1.5E+07	3.3E+07	1800	30	0.0E+00
	Sandstone		34	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	Clay		13	9	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		16	5	5.0E+07	1.0E+08	1850	28	2.0E+04
	SandyClay		7	7	2.5E+07	5.0E+07	1850	30	1.0E+04
	Clay		9	6	2.0E+07	4.0E+07	1900	22	1.0E+04
	Sand		20	3	6.0E+07	1.2E+08	1800	36	0.0E+00
	SiltyClay		18	6	2.0E+07	4.0E+07	2000	25	2.0E+04
	Clay		18	3	3.5E+07	7.0E+07	1900	22	3.0E+04
	Sandstone		17	5	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 3	10	4	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		5	2	1.6E+07	3.5E+07	1850	26	1.0E+04
	Sand		4	6	1.5E+07	3.3E+07	1800	30	0.0E+00
	Sandstone		34	4	1.0E+08	2.0E+08	2000	40	1.0E+06
	Clay		13	3	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		16	1	5.0E+07	1.0E+08	1850	28	2.0E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

		SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 2	SandyClay	Point - 3	7	2.5E+07	5.0E+07	1850	30	1.0E+04
	Clay		9	2.0E+07	4.0E+07	1900	22	1.0E+04
	Sand		20	6.0E+07	1.2E+08	1800	36	0.0E+00
	SiltyClay		18	2.0E+07	4.0E+07	2000	25	2.0E+04
	Clay		18	3.5E+07	7.0E+07	1900	22	3.0E+04
	Sandstone		17	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 4	10	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		5	1.6E+07	3.5E+07	1850	26	1.0E+04
	Sand		4	1.5E+07	3.3E+07	1800	30	0.0E+00
	Sandstone		34	1.0E+08	2.0E+08	2000	40	1.0E+06
	Clay		13	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		16	5.0E+07	1.0E+08	1850	28	2.0E+04
	SandyClay		7	2.5E+07	5.0E+07	1850	30	1.0E+04
	Clay		9	2.0E+07	4.0E+07	1900	22	1.0E+04
	Sand		20	6.0E+07	1.2E+08	1800	36	0.0E+00
	SiltyClay		18	2.0E+07	4.0E+07	2000	25	2.0E+04
	Clay		18	3.5E+07	7.0E+07	1900	22	3.0E+04
	Sandstone		17	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 5	10	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		5	1.6E+07	3.5E+07	1850	26	1.0E+04
	Sand		4	1.5E+07	3.3E+07	1800	30	0.0E+00
	Sandstone		34	1.0E+08	2.0E+08	2000	40	1.0E+06
	Clay		13	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		16	5.0E+07	1.0E+08	1850	28	2.0E+04
	SandyClay		7	2.5E+07	5.0E+07	1850	30	1.0E+04
	Clay		9	2.0E+07	4.0E+07	1900	22	1.0E+04
	Sand		20	6.0E+07	1.2E+08	1800	36	0.0E+00
	SiltyClay		18	2.0E+07	4.0E+07	2000	25	2.0E+04
	Clay		18	3.5E+07	7.0E+07	1900	22	3.0E+04
	Sandstone		17	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
ZONE 3	ArtificialFill	Point - 1	13	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		4	1.3E+07	2.5E+07	1850	25	4.0E+03
	ClayeySand		2	2.0E+07	4.0E+07	1850	25	7.0E+03
	Clay		9	1.0E+07	2.0E+07	1900	20	1.0E+04
	SandyClay		8	1.5E+07	3.0E+07	1850	26	5.0E+03
	Clay		8	1.0E+07	2.0E+07	1900	20	1.0E+04
	Claystone		7	2.0E+07	4.0E+07	2100	30	1.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 2	13	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		4	1.3E+07	2.5E+07	1850	25	4.0E+03
	ClayeySand		2	2.0E+07	4.0E+07	1850	25	7.0E+03
	Clay		9	1.0E+07	2.0E+07	1900	20	1.0E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

		SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 3	SandyClay	Point - 2	8	1.5E+07	3.0E+07	1850	26	5.0E+03
	Clay		8	1.0E+07	2.0E+07	1900	20	1.0E+04
	Claystone		7	2.0E+07	4.0E+07	2100	30	1.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 3	13	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		4	1.3E+07	2.5E+07	1850	25	4.0E+03
	ClayeySand		2	2.0E+07	4.0E+07	1850	25	7.0E+03
	Clay		9	1.0E+07	2.0E+07	1900	20	1.0E+04
	SandyClay		8	1.5E+07	3.0E+07	1850	26	5.0E+03
	Clay		8	1.0E+07	2.0E+07	1900	20	1.0E+04
	Claystone		7	2.0E+07	4.0E+07	2100	30	1.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 4	13	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		4	1.3E+07	2.5E+07	1850	25	4.0E+03
	ClayeySand		2	2.0E+07	4.0E+07	1850	25	7.0E+03
	Clay		9	1.0E+07	2.0E+07	1900	20	1.0E+04
	SandyClay		8	1.5E+07	3.0E+07	1850	26	5.0E+03
	Clay		8	1.0E+07	2.0E+07	1900	20	1.0E+04
	Claystone		7	2.0E+07	4.0E+07	2100	30	1.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 5	13	3.0E+06	5.0E+06	1700	20	1.0E+04
	SandyClay		4	1.3E+07	2.5E+07	1850	25	4.0E+03
	ClayeySand		2	2.0E+07	4.0E+07	1850	25	7.0E+03
	Clay		9	1.0E+07	2.0E+07	1900	20	1.0E+04
	SandyClay		8	1.5E+07	3.0E+07	1850	26	5.0E+03
	Clay		8	1.0E+07	2.0E+07	1900	20	1.0E+04
	Claystone		7	2.0E+07	4.0E+07	2100	30	1.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
ZONE 4	ArtificialFill	Point - 1	18	1.5E+07	3.3E+07	1900	22	1.5E+04
	SandyClay		7	6.0E+07	1.2E+08	1850	30	2.0E+04
	Clay		9	1.8E+07	3.6E+07	1900	22	1.5E+04
	ClayeySand		3	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		13	3.0E+07	6.0E+07	1900	22	3.0E+04
	SandyClay		4	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		7	2.0E+07	4.0E+07	1900	22	1.5E+04
	Claystone		19	3.0E+07	6.0E+07	2100	35	3.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 2	18	1.5E+07	3.3E+07	1900	22	1.5E+04
	SandyClay		7	6.0E+07	1.2E+08	1850	30	2.0E+04
	Clay		9	1.8E+07	3.6E+07	1900	22	1.5E+04
	ClayeySand		3	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		13	3.0E+07	6.0E+07	1900	22	3.0E+04
	SandyClay		4	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		7	2.0E+07	4.0E+07	1900	22	1.5E+04
	Claystone		19	3.0E+07	6.0E+07	2100	35	3.0E+04
	Rock		24	5.0E+08	1.0E+09	2800	42	1.0E+07

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

			SPT- N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 4	ArtificialFill	Point - 3	18	3	1.5E+07	3.3E+07	1900	22	1.5E+04
	SandyClay		7	1	6.0E+07	1.2E+08	1850	30	2.0E+04
	Clay		9	5	1.8E+07	3.6E+07	1900	22	1.5E+04
	ClayeySand		3	6	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		13	1	3.0E+07	6.0E+07	1900	22	3.0E+04
	SandyClay		4	8	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		7	14	2.0E+07	4.0E+07	1900	22	1.5E+04
	Claystone		19	2	3.0E+07	6.0E+07	2100	35	3.0E+04
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 4	18	4	1.5E+07	3.3E+07	1900	22	1.5E+04
	SandyClay		7	5	6.0E+07	1.2E+08	1850	30	2.0E+04
	Clay		9	4	1.8E+07	3.6E+07	1900	22	1.5E+04
	ClayeySand		3	1	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		13	6	3.0E+07	6.0E+07	1900	22	3.0E+04
	SandyClay		4	15	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		7	3	2.0E+07	4.0E+07	1900	22	1.5E+04
	Claystone		19	2	3.0E+07	6.0E+07	2100	35	3.0E+04
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 5	18	1	1.5E+07	3.3E+07	1900	22	1.5E+04
	SandyClay		7	4	6.0E+07	1.2E+08	1850	30	2.0E+04
	Clay		9	6	1.8E+07	3.6E+07	1900	22	1.5E+04
	ClayeySand		3	8	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		13	9	3.0E+07	6.0E+07	1900	22	3.0E+04
	SandyClay		4	3	2.0E+07	4.0E+07	1850	28	1.0E+04
	Clay		7	2	2.0E+07	4.0E+07	1900	22	1.5E+04
	Claystone		19	7	3.0E+07	6.0E+07	2100	35	3.0E+04
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
ZONE 5	ArtificialFill	Point - 1	35	1	5.0E+06	1.0E+07	1700	20	1.5E+04
	SandyClay		4	8	1.3E+07	2.5E+07	1850	25	4.0E+03
	Clay		3	5	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sandstone		24	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	ClayeySand		23	5	7.0E+07	1.4E+08	1850	30	2.0E+04
	Clay		19	8	5.0E+07	1.0E+08	1900	25	4.0E+04
	SandyClay		10	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		4	2	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		7	6	2.0E+07	4.0E+07	2100	35	5.0E+04
	Sandstone		19	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 2	35	1	5.0E+06	1.0E+07	1700	20	1.5E+04
	SandyClay		4	1	1.3E+07	2.5E+07	1850	25	4.0E+03
	Clay		3	1	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sandstone		24	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	ClayeySand		23	8	7.0E+07	1.4E+08	1850	30	2.0E+04
	Clay		19	11	5.0E+07	1.0E+08	1900	25	4.0E+04
	SandyClay		10	7	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		4	3	1.0E+07	2.0E+07	1900	22	1.0E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

			SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 5	Claystone	Point - 2	7	6	2.0E+07	4.0E+07	2100	35	5.0E+04
	Sandstone		19	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 3	35	4	5.0E+06	1.0E+07	1700	20	1.5E+04
	SandyClay		4	1	1.3E+07	2.5E+07	1850	25	4.0E+03
	Clay		3	3	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sandstone		24	1	1.0E+08	2.0E+08	2000	40	1.0E+06
	ClayeySand		23	6	7.0E+07	1.4E+08	1850	30	2.0E+04
	Clay		19	6	5.0E+07	1.0E+08	1900	25	4.0E+04
	SandyClay		10	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		4	3	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		7	8	2.0E+07	4.0E+07	2100	35	5.0E+04
	Sandstone		19	10	1.0E+08	2.0E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 4	35	5	5.0E+06	1.0E+07	1700	20	1.5E+04
	SandyClay		4	1	1.3E+07	2.5E+07	1850	25	4.0E+03
	Clay		3	6	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sandstone		24	3	1.0E+08	2.0E+08	2000	40	1.0E+06
	ClayeySand		23	1	7.0E+07	1.4E+08	1850	30	2.0E+04
	Clay		19	8	5.0E+07	1.0E+08	1900	25	4.0E+04
	SandyClay		10	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		4	3	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		7	3	2.0E+07	4.0E+07	2100	35	5.0E+04
	Sandstone		19	10	1.0E+08	2.0E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
	ArtificialFill	Point - 5	35	2	5.0E+06	1.0E+07	1700	20	1.5E+04
	SandyClay		4	3	1.3E+07	2.5E+07	1850	25	4.0E+03
	Clay		3	8	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sandstone		24	2	1.0E+08	2.0E+08	2000	40	1.0E+06
	ClayeySand		23	4	7.0E+07	1.4E+08	1850	30	2.0E+04
	Clay		19	2	5.0E+07	1.0E+08	1900	25	4.0E+04
	SandyClay		10	7	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		4	3	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		7	2	2.0E+07	4.0E+07	2100	35	5.0E+04
	Sandstone		19	7	1.0E+08	2.0E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+07
ZONE 6	ArtificialFill	Point - 1	2	7	2.0E+06	4.0E+06	1700	20	5.0E+03
	SandyClay		15	2	5.0E+07	1.0E+08	1850	30	1.5E+04
	SiltyClay		10	9	1.5E+07	3.0E+07	2000	25	1.8E+04
	ClayeySand		21	3	7.0E+07	1.4E+08	1850	30	2.3E+04
	Clay		18	2	4.0E+07	8.0E+07	1900	24	3.0E+04
	Sand		6	1	2.0E+07	4.0E+07	1800	32	0.0E+00
	Clay		4	1	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		14	10	4.0E+07	8.0E+07	2100	35	5.0E+04
	Gravel		7	5	5.0E+07	1.0E+08	2000	34	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

		SPT- N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)	
ZONE 6	ArtificialFill	Point - 2	2	5	2.0E+06	4.0E+06	1700	20	5.0E+03
	SandyClay		15	5	5.0E+07	1.0E+08	1850	30	1.5E+04
	SiltyClay		10	14	1.5E+07	3.0E+07	2000	25	1.8E+04
	ClayeySand		21	4	7.0E+07	1.4E+08	1850	30	2.3E+04
	Clay		18	1	4.0E+07	8.0E+07	1900	24	3.0E+04
	Sand		6	2	2.0E+07	4.0E+07	1800	32	0.0E+00
	Clay		4	1	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		14	4	4.0E+07	8.0E+07	2100	35	5.0E+04
	Gravel		7	4	5.0E+07	1.0E+08	2000	34	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 3	2	4	2.0E+06	4.0E+06	1700	20	5.0E+03
	SandyClay		15	9	5.0E+07	1.0E+08	1850	30	1.5E+04
	SiltyClay		10	5	1.5E+07	3.0E+07	2000	25	1.8E+04
	ClayeySand		21	8	7.0E+07	1.4E+08	1850	30	2.3E+04
	Clay		18	3	4.0E+07	8.0E+07	1900	24	3.0E+04
	Sand		6	1	2.0E+07	4.0E+07	1800	32	0.0E+00
	Clay		4	1	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		14	5	4.0E+07	8.0E+07	2100	35	5.0E+04
	Gravel		7	1	5.0E+07	1.0E+08	2000	34	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 4	2	1	2.0E+06	4.0E+06	1700	20	5.0E+03
	SandyClay		15	1	5.0E+07	1.0E+08	1850	30	1.5E+04
	SiltyClay		10	2	1.5E+07	3.0E+07	2000	25	1.8E+04
	ClayeySand		21	9	7.0E+07	1.4E+08	1850	30	2.3E+04
	Clay		18	1	4.0E+07	8.0E+07	1900	24	3.0E+04
	Sand		6	6	2.0E+07	4.0E+07	1800	32	0.0E+00
	Clay		4	12	1.0E+07	2.0E+07	1900	22	1.0E+04
	Claystone		14	7	4.0E+07	8.0E+07	2100	35	5.0E+04
	Gravel		7	1	5.0E+07	1.0E+08	2000	34	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
ArtificialFill	Point - 5	2	8	2.0E+06	4.0E+06	1700	20	5.0E+03	
SandyClay		15	1	5.0E+07	1.0E+08	1850	30	1.5E+04	
SiltyClay		10	1	1.5E+07	3.0E+07	2000	25	1.8E+04	
ClayeySand		21	8	7.0E+07	1.4E+08	1850	30	2.3E+04	
Clay		18	3	4.0E+07	8.0E+07	1900	24	3.0E+04	
Sand		6	3	2.0E+07	4.0E+07	1800	32	0.0E+00	
Clay		4	2	1.0E+07	2.0E+07	1900	22	1.0E+04	
Claystone		14	10	4.0E+07	8.0E+07	2100	35	5.0E+04	
Gravel		7	2	5.0E+07	1.0E+08	2000	34	0.0E+00	
Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06	
ZONE 7	ArtificialFill	Point - 1	57	3	1.0E+07	2.0E+07	1700	22	2.0E+04
	SandyClay		17	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Gravel		2	2	1.5E+07	3.0E+07	2000	25	0.0E+00
	SiltyClay		8	14	1.5E+07	3.0E+07	2000	24	1.7E+04
	ClayeySand		19	4	5.5E+07	1.1E+08	1850	28	2.5E+04
	Clay		6	1	1.0E+07	2.0E+07	1900	22	2.0E+04

Table B.1 (continued): Calculated and defined soil properties for FDM models.

			SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 7	Sand	Point - 1	5	3	2.0E+07	4.0E+07	1800	30	0.0E+00
	Claystone		4	4	1.5E+07	3.0E+07	2100	32	2.0E+04
	Gravel		3	4	2.0E+07	4.0E+07	2000	30	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 2	57	3	1.0E+07	2.0E+07	1700	22	2.0E+04
	SandyClay		17	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Gravel		2	2	1.5E+07	3.0E+07	2000	25	0.0E+00
	SiltyClay		8	14	1.5E+07	3.0E+07	2000	24	1.7E+04
	ClayeySand		19	4	5.5E+07	1.1E+08	1850	28	2.5E+04
	Clay		6	1	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sand		5	3	2.0E+07	4.0E+07	1800	30	0.0E+00
	Claystone		4	4	1.5E+07	3.0E+07	2100	32	2.0E+04
	Gravel		3	4	2.0E+07	4.0E+07	2000	30	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 3	57	3	1.0E+07	2.0E+07	1700	22	2.0E+04
	SandyClay		17	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Gravel		2	2	1.5E+07	3.0E+07	2000	25	0.0E+00
	SiltyClay		8	14	1.5E+07	3.0E+07	2000	24	1.7E+04
	ClayeySand		19	4	5.5E+07	1.1E+08	1850	28	2.5E+04
	Clay		6	1	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sand		5	3	2.0E+07	4.0E+07	1800	30	0.0E+00
	Claystone		4	4	1.5E+07	3.0E+07	2100	32	2.0E+04
	Gravel		3	4	2.0E+07	4.0E+07	2000	30	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 4	57	3	1.0E+07	2.0E+07	1700	22	2.0E+04
	SandyClay		17	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Gravel		2	2	1.5E+07	3.0E+07	2000	25	0.0E+00
	SiltyClay		8	14	1.5E+07	3.0E+07	2000	24	1.7E+04
	ClayeySand		19	4	5.5E+07	1.1E+08	1850	28	2.5E+04
	Clay		6	1	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sand		5	3	2.0E+07	4.0E+07	1800	30	0.0E+00
	Claystone		4	4	1.5E+07	3.0E+07	2100	32	2.0E+04
	Gravel		3	4	2.0E+07	4.0E+07	2000	30	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 5	57	3	1.0E+07	2.0E+07	1700	22	2.0E+04
	SandyClay		17	3	5.0E+07	1.0E+08	1850	30	1.5E+04
	Gravel		2	2	1.5E+07	3.0E+07	2000	25	0.0E+00
	SiltyClay		8	14	1.5E+07	3.0E+07	2000	24	1.7E+04
	ClayeySand		19	4	5.5E+07	1.1E+08	1850	28	2.5E+04
	Clay		6	1	1.0E+07	2.0E+07	1900	22	2.0E+04
	Sand		5	3	2.0E+07	4.0E+07	1800	30	0.0E+00
	Claystone		4	4	1.5E+07	3.0E+07	2100	32	2.0E+04
	Gravel		3	4	2.0E+07	4.0E+07	2000	30	0.0E+00
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
ZONE 8	ArtificialFill	Point - 1	20	4	5.0E+06	1.0E+07	1700	22	1.5E+04
	GravellySilt		19	4	5.0E+07	1.0E+08	2050	30	1.0E+04
	Clay		2	4	1.0E+07	2.0E+07	1900	22	1.0E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

		SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 8	Sandyclay	10	6	5.0E+07	1.0E+08	1850	28	1.0E+04
	Clay	12	2	3.0E+07	6.0E+07	1900	22	2.0E+04
	SiltyClay	4	9	1.0E+07	2.0E+07	2000	25	1.0E+04
	Gravel	19	2	1.0E+08	2.0E+08	2000	34	0.0E+00
	Limestone	20	4	2.0E+09	4.0E+09	2700	35	1.0E+06
	Rock	24	5	1.0E+09	2.0E+09	2800	42	1.0E+06
	ArtificialFill	20	4	5.0E+06	1.0E+07	1700	22	1.5E+04
	GravellySilt	19	4	5.0E+07	1.0E+08	2050	30	1.0E+04
	Clay	2	4	1.0E+07	2.0E+07	1900	22	1.0E+04
	Sandyclay	10	6	5.0E+07	1.0E+08	1850	28	1.0E+04
	Clay	12	2	3.0E+07	6.0E+07	1900	22	2.0E+04
	SiltyClay	4	9	1.0E+07	2.0E+07	2000	25	1.0E+04
	Gravel	19	2	1.0E+08	2.0E+08	2000	34	0.0E+00
	Limestone	20	4	2.0E+09	4.0E+09	2700	35	1.0E+06
	Rock	24	5	1.0E+09	2.0E+09	2800	42	1.0E+06
	ArtificialFill	20	4	5.0E+06	1.0E+07	1700	22	1.5E+04
	GravellySilt	19	2	5.0E+07	1.0E+08	2050	30	1.0E+04
	Clay	2	6	1.0E+07	2.0E+07	1900	22	1.0E+04
	Sandyclay	10	8	5.0E+07	1.0E+08	1850	28	1.0E+04
	Clay	12	6	3.0E+07	6.0E+07	1900	22	2.0E+04
	SiltyClay	4	3	1.0E+07	2.0E+07	2000	25	1.0E+04
	Gravel	19	2	1.0E+08	2.0E+08	2000	34	0.0E+00
	Limestone	20	4	2.0E+09	4.0E+09	2700	35	1.0E+06
	Rock	24	5	1.0E+09	2.0E+09	2800	42	1.0E+06
	ArtificialFill	20	4	5.0E+06	1.0E+07	1700	22	1.5E+04
	GravellySilt	19	2	5.0E+07	1.0E+08	2050	30	1.0E+04
	Clay	2	6	1.0E+07	2.0E+07	1900	22	1.0E+04
	Sandyclay	10	8	5.0E+07	1.0E+08	1850	28	1.0E+04
	Clay	12	6	3.0E+07	6.0E+07	1900	22	2.0E+04
	SiltyClay	4	3	1.0E+07	2.0E+07	2000	25	1.0E+04
	Gravel	19	2	1.0E+08	2.0E+08	2000	34	0.0E+00
	Limestone	20	4	2.0E+09	4.0E+09	2700	35	1.0E+06
	Rock	24	5	1.0E+09	2.0E+09	2800	42	1.0E+06
	ArtificialFill	20	4	5.0E+06	1.0E+07	1700	22	1.5E+04
	GravellySilt	19	4	5.0E+07	1.0E+08	2050	30	1.0E+04
	Clay	2	4	1.0E+07	2.0E+07	1900	22	1.0E+04
	Sandyclay	10	6	5.0E+07	1.0E+08	1850	28	1.0E+04
	Clay	12	2	3.0E+07	6.0E+07	1900	22	2.0E+04
	SiltyClay	4	9	1.0E+07	2.0E+07	2000	25	1.0E+04
	Gravel	19	2	1.0E+08	2.0E+08	2000	34	0.0E+00
	Limestone	20	4	2.0E+09	4.0E+09	2700	35	1.0E+06
	Rock	24	5	1.0E+09	2.0E+09	2800	42	1.0E+06
ZONE 9	ArtificialFill	15	1	3.0E+06	5.0E+06	1700	20	1.0E+04
	GravellySilt	20	1	5.0E+07	1.0E+08	2050	30	1.0E+04
	Sandyclay	9	16	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay	12	2	3.0E+07	6.0E+07	1900	22	3.0E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

			SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 9	ClayeySand	Point - 2	7	13	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone		20	3	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 2	15	1	3.0E+06	5.0E+06	1700	20	1.0E+04
	GravellySilt		20	1	5.0E+07	1.0E+08	2050	30	1.0E+04
	Sandyclay		9	9	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay	Point - 2	12	9	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		7	12	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone		20	3	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock	Point - 2	24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill		15	4	3.0E+06	5.0E+06	1700	20	1.0E+04
	GravellySilt		20	8	5.0E+07	1.0E+08	2050	30	1.0E+04
	Sandyclay	Point - 3	9	6	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		12	2	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		7	13	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone	Point - 3	20	3	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill		15	4	3.0E+06	5.0E+06	1700	20	1.0E+04
	GravellySilt	Point - 4	20	8	5.0E+07	1.0E+08	2050	30	1.0E+04
	Sandyclay		9	6	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay		12	2	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand	Point - 4	7	13	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone		20	3	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock		24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
	ArtificialFill	Point - 5	15	2	3.0E+06	5.0E+06	1700	20	1.0E+04
	GravellySilt		20	6	5.0E+07	1.0E+08	2050	30	1.0E+04
	Sandyclay		9	6	5.0E+07	1.0E+08	1850	30	1.5E+04
	Clay	Point - 5	12	2	3.0E+07	6.0E+07	1900	22	3.0E+04
	ClayeySand		7	13	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone		20	3	8.0E+07	1.6E+08	2000	40	1.0E+06
	Rock	Point - 5	24	5	5.0E+08	1.0E+09	2800	42	1.0E+06
ZONE 10	ArtificialFill	Point - 1	14	3	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay		15	9	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay		10	8	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand		13	1	4.0E+07	8.0E+07	1800	35	0.0E+00
	ClayeySand		7	12	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone	Point - 1	9	3	8.0E+07	1.6E+08	2000	40	0.0E+00
	Rock		24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill		14	7	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay		15	1	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay		10	1	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand	Point - 2	13	1	4.0E+07	8.0E+07	1800	35	0.0E+00
	ClayeySand		7	15	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone		9	11	8.0E+07	1.6E+08	2000	40	0.0E+00
	Rock		24	5	1.0E+10	2.0E+10	2800	42	1.0E+06

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

		SPT- N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 10	ArtificialFill	14	4	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	15	8	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay	10	5	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand	13	1	4.0E+07	8.0E+07	1800	35	0.0E+00
	ClayeySand	7	1	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone	9	1	8.0E+07	1.6E+08	2000	40	0.0E+00
	Rock	24	20	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	14	2	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	15	1	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay	10	13	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand	13	1	4.0E+07	8.0E+07	1800	35	0.0E+00
	ClayeySand	7	1	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone	9	2	8.0E+07	1.6E+08	2000	40	0.0E+00
	Rock	24	21	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	14	3	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	15	9	4.0E+07	8.0E+07	1850	30	1.5E+04
	Clay	10	9	3.0E+07	6.0E+07	1900	22	3.0E+04
	Sand	13	2	4.0E+07	8.0E+07	1800	35	0.0E+00
ZONE 11	ClayeySand	7	7	5.0E+07	1.0E+08	1850	28	2.0E+04
	Sandstone	9	3	8.0E+07	1.6E+08	2000	40	0.0E+00
	Rock	24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	30	3	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	9	8	6.0E+07	1.2E+08	1850	30	1.5E+04
	SiltyClay	11	8	1.5E+07	3.2E+07	2000	25	1.5E+04
	SandyClay	13	2	4.5E+07	9.0E+07	1850	30	1.5E+04
	Gravel	10	9	9.0E+07	2.0E+08	2000	34	0.0E+00
	Sandstone	19	7	5.0E+07	1.0E+08	2000	40	0.0E+00
	Rock	24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	30	2	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	9	1	6.0E+07	1.2E+08	1850	30	1.5E+04
	SiltyClay	11	9	1.5E+07	3.2E+07	2000	25	1.5E+04
	SandyClay	13	9	4.5E+07	9.0E+07	1850	30	1.5E+04
	Gravel	10	9	9.0E+07	2.0E+08	2000	34	0.0E+00
	Sandstone	19	7	5.0E+07	1.0E+08	2000	40	0.0E+00
	Rock	24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	30	7	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	9	4	6.0E+07	1.2E+08	1850	30	1.5E+04
	SiltyClay	11	1	1.5E+07	3.2E+07	2000	25	1.5E+04
	SandyClay	13	20	4.5E+07	9.0E+07	1850	30	1.5E+04
	Gravel	10	1	9.0E+07	2.0E+08	2000	34	0.0E+00
	Sandstone	19	4	5.0E+07	1.0E+08	2000	40	0.0E+00
	Rock	24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	30	2	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay	9	1	6.0E+07	1.2E+08	1850	30	1.5E+04
	SiltyClay	11	7	1.5E+07	3.2E+07	2000	25	1.5E+04
	SandyClay	13	9	4.5E+07	9.0E+07	1850	30	1.5E+04

Table B.1 (continued) : Calculated and defined soil properties for FDM models.

			SPT-N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (Degree)	Cohesion (Pa)
ZONE 11	Gravel	Point - 4	10	11	9.0E+07	2.0E+08	2000	34	0.0E+00
	Sandstone		19	7	5.0E+07	1.0E+08	2000	40	0.0E+00
	Rock		24	5	1.0E+10	2.0E+10	2800	42	1.0E+06
	ArtificialFill	Point - 5	30	3	5.0E+06	1.0E+07	1700	20	1.0E+04
	SandyClay		9	2	6.0E+07	1.2E+08	1850	30	1.5E+04
	SiltyClay		11	14	1.5E+07	3.2E+07	2000	25	1.5E+04
	SandyClay		13	2	4.5E+07	9.0E+07	1850	30	1.5E+04
	Gravel		10	9	9.0E+07	2.0E+08	2000	34	0.0E+00
	Sandstone		19	7	5.0E+07	1.0E+08	2000	40	0.0E+00
	Rock		24	8	1.0E+10	2.0E+10	2800	42	1.0E+06

APPENDIX C : Finn constants calculated for all zones in FDM models.

Table C.1: Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 1	ArtificialFill-1	Point - 1		
	Sand-1b		0.32	0.62
	SiltySand-1		0.15	1.36
	ClayeySand-1		0.24	0.82
	SandyClay-1		1.83	0.19
	Clay-1			
	Sand-1a		0.12	1.75
	SiltyClay-1			
	Claystone-1			
	Sandstone-1		0.13	1.59
	Rock-1			
	ArtificialFill-1	Point - 2		
	Sand-1b		0.32	0.62
	SiltySand-1		0.15	1.36
	ClayeySand-1		0.24	0.82
	SandyClay-1		1.83	0.19
	Clay-1			
	Sand-1a		0.12	1.75
	SiltyClay-1			
	Claystone-1			
	Sandstone-1		0.13	1.59
	Rock-1			
	ArtificialFill-1	Point - 3		
	Sand-1b		0.32	0.62
	SiltySand-1		0.15	1.36
	ClayeySand-1		0.24	0.82
	SandyClay-1		1.83	0.19
	Clay-1			
	Sand-1a		0.12	1.75
	SiltyClay-1			
	Claystone-1			
	Sandstone-1		0.13	1.59
	Rock-1			
	ArtificialFill-1	Point - 4		
	Sand-1b		0.32	0.62
	SiltySand-1		0.15	1.36
	ClayeySand-1		0.24	0.82
	SandyClay-1		1.83	0.19
	Clay-1			
	Sand-1a		0.12	1.75
	SiltyClay-1			
	Claystone-1			
	Sandstone-1		0.13	1.59
	Rock-1			
	ArtificialFill-1	Point - 5		
	Sand-1b		0.32	0.62
	SiltySand-1		0.15	1.36
	ClayeySand-1		0.24	0.82
	SandyClay-1		1.83	0.19

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 1	Clay-1	Point - 5		
	Sand-1a		0.12	1.75
	SiltyClay-1			
	Claystone-1			
	Sandstone-1		0.13	1.59
	Rock-1			
ZONE 2	ArtificialFill	Point - 1		
	SandyClay		0.58	0.34
	Sand		0.77	0.27
	Sandstone		0.53	3.77
	Clay			
	ClayeySand		0.14	1.47
	SandyClay		0.38	0.52
	Clay			
	Sand		0.13	1.94
	SiltyClay			
	Clay			
	Sandstone		0.13	1.59
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.58	0.34
	Sand		0.77	0.27
	Sandstone		0.53	3.77
	Clay			
	ClayeySand		0.14	1.47
	SandyClay		0.38	0.52
	Clay			
	Sand		0.13	1.94
	SiltyClay			
	Clay			
	Sandstone		0.13	1.59
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.58	0.34
	Sand		0.77	0.27
	Sandstone		0.53	3.77
	Clay			
	ClayeySand		0.14	1.47
	SandyClay		0.38	0.52
	Clay			
	Sand		0.13	1.94
	SiltyClay			
	Clay			
	Sandstone		0.13	1.59
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.58	0.34
	Sand		0.77	0.27
	Sandstone		0.53	3.77
	Clay			
	ClayeySand		0.14	1.47
	SandyClay		0.38	0.52

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 2	Clay	Point - 4		
	Sand		0.13	1.94
	SiltyClay			
	Clay			
	Sandstone		0.13	1.59
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.58	0.34
	Sand		0.77	0.27
	Sandstone		0.53	3.77
	Clay			
	ClayeySand		0.14	1.47
	SandyClay		0.38	0.52
	Clay			
	Sand		0.13	1.94
	SiltyClay			
	Clay			
	Sandstone		0.13	1.59
	Rock			
ZONE 3	ArtificialFill	Point - 1		
	SandyClay		0.77	0.27
	ClayeySand		1.83	0.19
	Clay			
	SandyClay		0.32	0.62
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.77	0.27
	ClayeySand		1.83	0.19
	Clay			
	SandyClay		0.32	0.62
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.77	0.27
	ClayeySand		1.83	0.19
	Clay			
	SandyClay		0.32	0.62
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.77	0.27
	ClayeySand		1.83	0.19
	Clay			
	SandyClay		0.32	0.62
	Clay			
	Claystone			
	Rock			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 4	ArtificialFill	Point - 5		
	SandyClay		0.77	0.27
	ClayeySand		1.83	0.19
	Clay			
	SandyClay		0.32	0.62
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 1		
	SandyClay		0.38	0.52
	Clay			
	ClayeySand		1.12	0.18
	Clay			
	SandyClay		0.77	0.27
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.38	0.52
	Clay			
	ClayeySand		1.12	0.18
	Clay			
	SandyClay		0.77	0.27
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.38	0.52
	Clay			
	ClayeySand		1.12	0.18
	Clay			
	SandyClay		0.77	0.27
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.38	0.52
	Clay			
	ClayeySand		1.12	0.18
	Clay			
	SandyClay		0.77	0.27
	Clay			
	Claystone			
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.38	0.52
	Clay			
	ClayeySand		1.12	0.18
	Clay			
	SandyClay		0.77	0.27
	Clay			
	Claystone			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
Z4	Rock	P5		
ZONE 5	ArtificialFill	Point - 1		
	SandyClay		0.77	0.27
	Clay			
	Sandstone		0.82	2.44
	ClayeySand		0.86	2.32
	Clay			
	SandyClay		0.24	0.82
	Clay			
	Claystone			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.77	0.27
	Clay			
	Sandstone		0.82	2.44
	ClayeySand		0.86	2.32
	Clay			
	SandyClay		0.24	0.82
	Clay			
	Claystone			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.77	0.27
	Clay			
	Sandstone		0.82	2.44
	ClayeySand		0.86	2.32
	Clay			
	SandyClay		0.24	0.82
	Clay			
	Claystone			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.77	0.27
	Clay			
	Sandstone		0.82	2.44
	ClayeySand		0.86	2.32
	Clay			
	SandyClay		0.24	0.82
	Clay			
	Claystone			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.77	0.27
	Clay			
	Sandstone		0.82	2.44
	ClayeySand		0.86	2.32
	Clay			
	SandyClay		0.24	0.82
	Clay			
	Claystone			
	Claystone			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
Z5	Rock	P5		
	Sandstone		0.20	1.82
ZONE 6	ArtificialFill	Point - 1		
	SandyClay		0.15	1.36
	SiltyClay			
	ClayeySand		0.97	2.67
	Clay			
	Sand		0.46	0.43
	Clay			
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.15	1.36
	SiltyClay			
	ClayeySand		0.97	2.67
	Clay			
	Sand		0.46	0.43
	Clay			
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.15	1.36
	SiltyClay			
	ClayeySand		0.97	2.67
	Clay			
	Sand		0.46	0.43
	Clay			
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.15	1.36
	SiltyClay			
	ClayeySand		0.97	2.67
	Clay			
	Sand		0.46	0.43
	Clay			
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.15	1.36
	SiltyClay			
	ClayeySand		0.97	2.67
	Clay			
	Sand		0.46	0.43
	Clay			
	Claystone			
	Gravel			
	Rock			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 7	ArtificialFill	Point - 1		
	SandyClay		0.13	1.59
	Gravel			
	SiltyClay			
	ClayeySand		0.20	1.82
	Clay			
	Sand		0.58	0.34
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.13	1.59
	Gravel			
	SiltyClay			
	ClayeySand		0.20	1.82
	Clay			
	Sand		0.58	0.34
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.13	1.59
	Gravel			
	SiltyClay			
	ClayeySand		0.20	1.82
	Clay			
	Sand		0.58	0.34
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.13	1.59
	Gravel			
	SiltyClay			
	ClayeySand		0.20	1.82
	Clay			
	Sand		0.58	0.34
	Claystone			
	Gravel			
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.13	1.59
	Gravel			
	SiltyClay			
	ClayeySand		0.20	1.82
	Clay			
	Sand		0.58	0.34
	Claystone			
	Gravel			
	Rock			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 8	ArtificialFill	Point - 1		
	GravellySilt			
	Clay			
	Sandyclay		0.24	0.82
	Clay			
	SiltyClay			
	Gravel			
	Limestone			
	Rock			
	ArtificialFill	Point - 2		
	GravellySilt			
	Clay			
	Sandyclay		0.24	0.82
	Clay			
	SiltyClay			
	Gravel			
	Limestone			
	Rock			
	ArtificialFill	Point - 3		
	GravellySilt			
	Clay			
	Sandyclay		0.24	0.82
	Clay			
	SiltyClay			
	Gravel			
	Limestone			
	Rock			
	ArtificialFill	Point - 4		
	GravellySilt			
	Clay			
	Sandyclay		0.24	0.82
	Clay			
	SiltyClay			
	Gravel			
	Limestone			
	Rock			
	ArtificialFill	Point - 5		
	GravellySilt			
	Clay			
	Sandyclay		0.24	0.82
	Clay			
	SiltyClay			
	Gravel			
	Limestone			
	Rock			
ZONE 9	ArtificialFill	Point - 1		
	GravellySilt			
	Sandyclay		0.28	0.72
	Clay			
	ClayeySand		0.38	0.52
	Sandstone		0.13	1.94
	Rock			

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 9	ArtificialFill	Point - 2		
	GravellySilt			
	Sandyclay		0.28	0.72
	Clay			
	ClayeySand		0.38	0.52
	Sandstone		0.13	1.94
	Rock			
	ArtificialFill	Point - 3		
	GravellySilt			
	Sandyclay		0.28	0.72
	Clay			
	ClayeySand		0.38	0.52
	Sandstone		0.13	1.94
	Rock			
	ArtificialFill	Point - 4		
	GravellySilt			
	Sandyclay		0.28	0.72
	Clay			
	ClayeySand		0.38	0.52
	Sandstone		0.13	1.94
	Rock			
	ArtificialFill	Point - 5		
	GravellySilt			
	Sandyclay		0.28	0.72
	Clay			
	ClayeySand		0.38	0.52
	Sandstone		0.13	1.94
	Rock			
ZONE 10	ArtificialFill	Point - 1		
	SandyClay		0.15	1.36
	Clay			
	Sand		0.18	1.13
	ClayeySand		0.38	0.52
	Sandstone		0.28	0.72
	Rock			
	ArtificialFill	Point - 2		
	SandyClay		0.15	1.36
	Clay			
	Sand		0.18	1.13
	ClayeySand		0.38	0.52
	Sandstone		0.28	0.72
	Rock			
	ArtificialFill	Point - 3		
	SandyClay		0.15	1.36
	Clay			
	Sand		0.18	1.13
	ClayeySand		0.38	0.52
	Sandstone		0.28	0.72
	Rock			
	ArtificialFill	Point - 4		
	SandyClay		0.15	1.36
	Clay			
	Sand		0.18	1.13
	ClayeySand		0.38	0.52

Table C.1 (continued) : Finn constants calculated for all zones in FDM models.

			Finn C1	Finn C2
ZONE 10	Sandstone	P4	0.28	0.72
	Rock			
	ArtificialFill	Point - 5		
	SandyClay		0.15	1.36
	Clay			
	Sand		0.18	1.13
	ClayeySand		0.38	0.52
	Sandstone		0.28	0.72
	Rock			
	ArtificialFill			
ZONE 11	SandyClay	Point - 1	0.28	0.72
	SiltyClay			
	SandyClay		0.18	1.13
	Gravel			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill			
	SandyClay	Point - 2	0.28	0.72
	SiltyClay			
	SandyClay		0.18	1.13
	Gravel			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill			
	SandyClay	Point - 3	0.28	0.72
	SiltyClay			
	SandyClay		0.18	1.13
	Gravel			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill			
	SandyClay	Point - 4	0.28	0.72
	SiltyClay			
	SandyClay		0.18	1.13
	Gravel			
	Sandstone		0.20	1.82
	Rock			
	ArtificialFill			
	SandyClay	Point - 5	0.28	0.72
	SiltyClay			
	SandyClay		0.18	1.13
	Gravel			
	Sandstone		0.20	1.82
	Rock			

APPENDIX D : Soil property groups for zone 1 to zone 11.

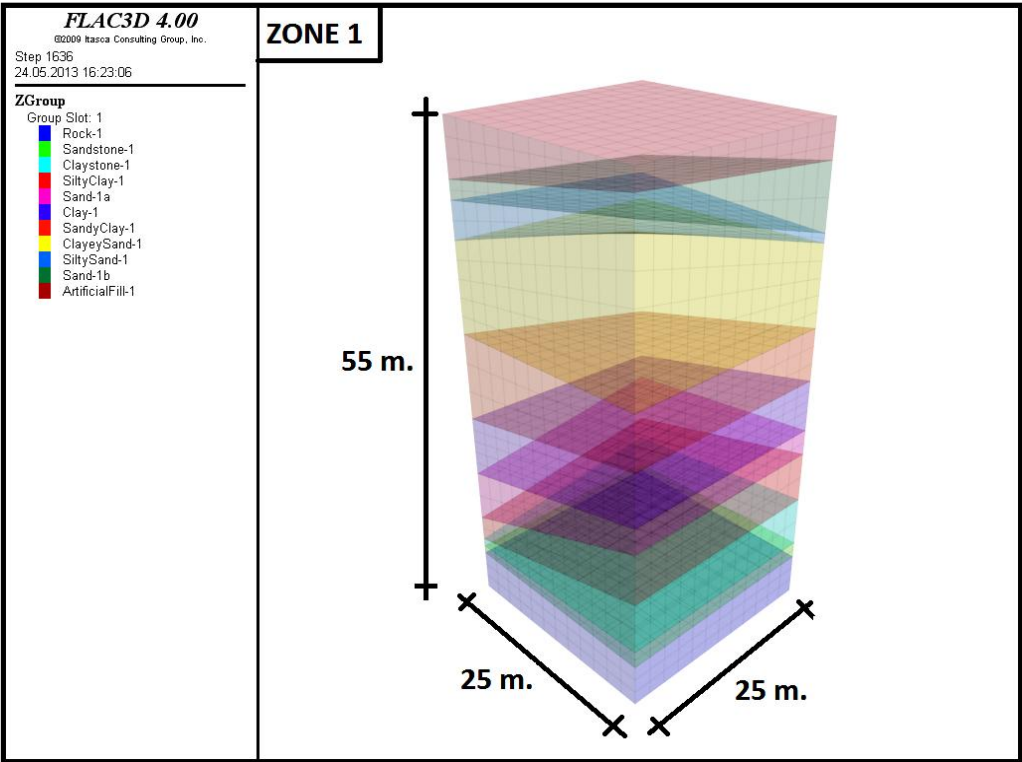


Figure D.1 : Soil property groups for zone 1.

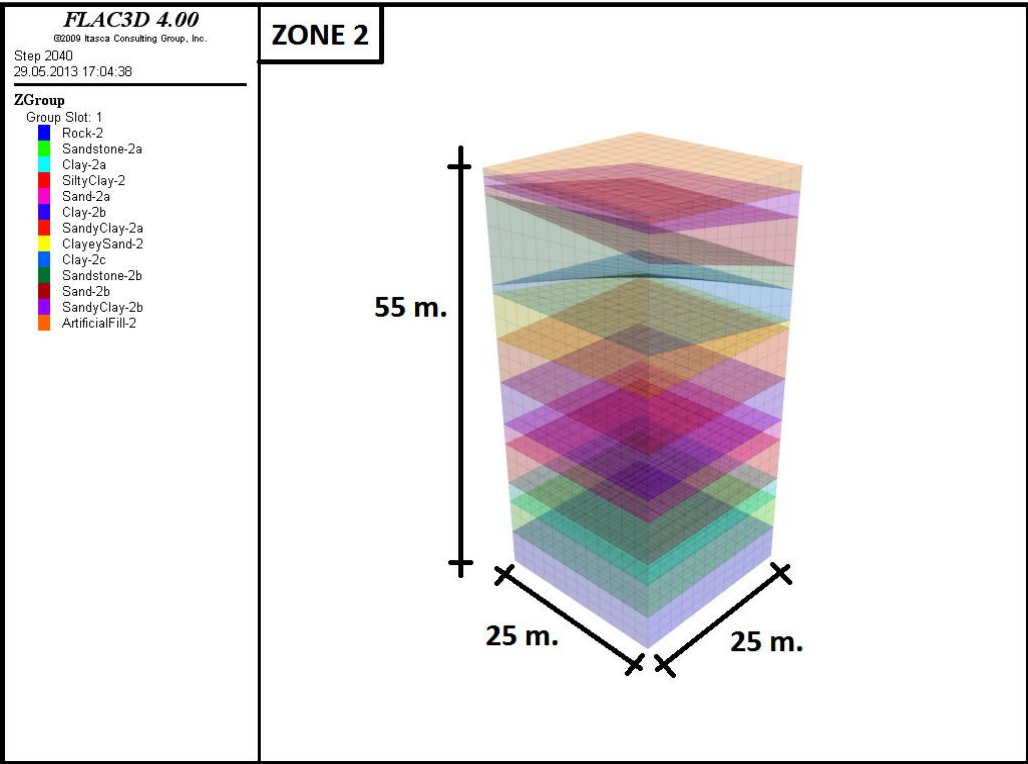


Figure D.2 : Soil property groups for zone 2.

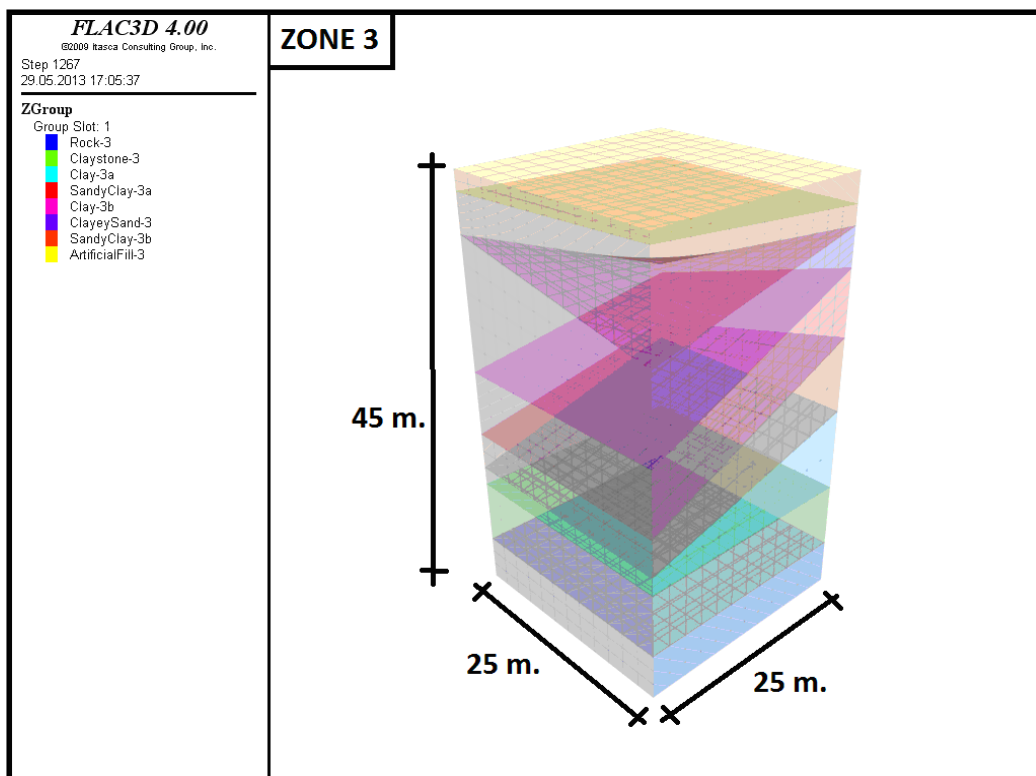


Figure D.3 : Soil property groups for zone 3.

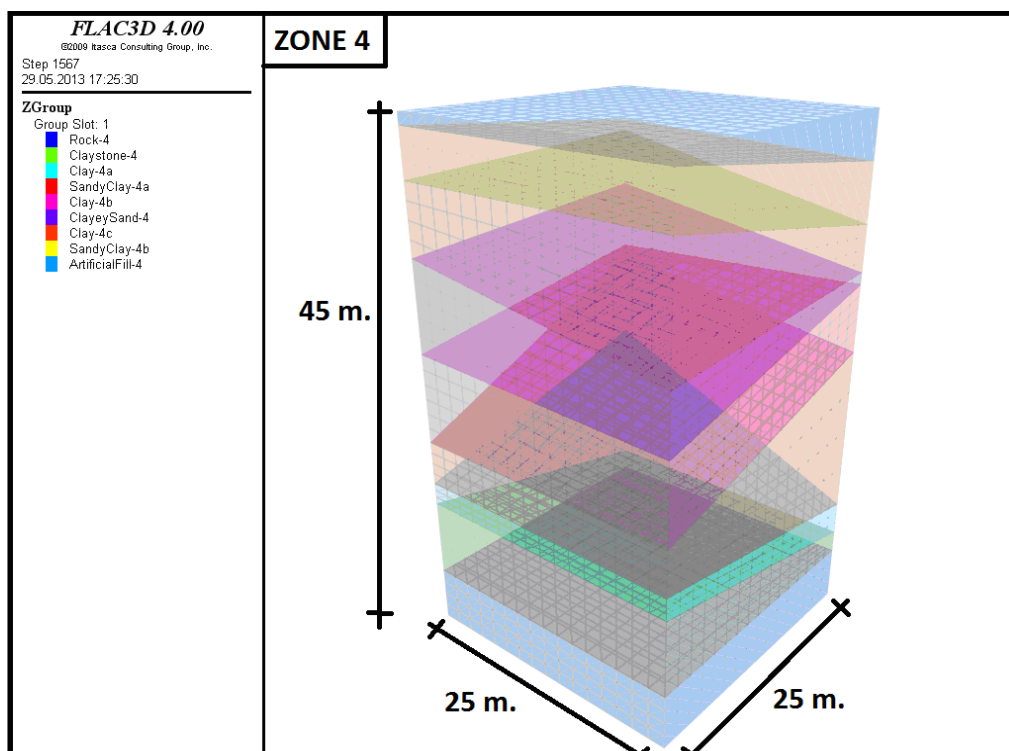


Figure D.4 : Soil property groups for zone 4.

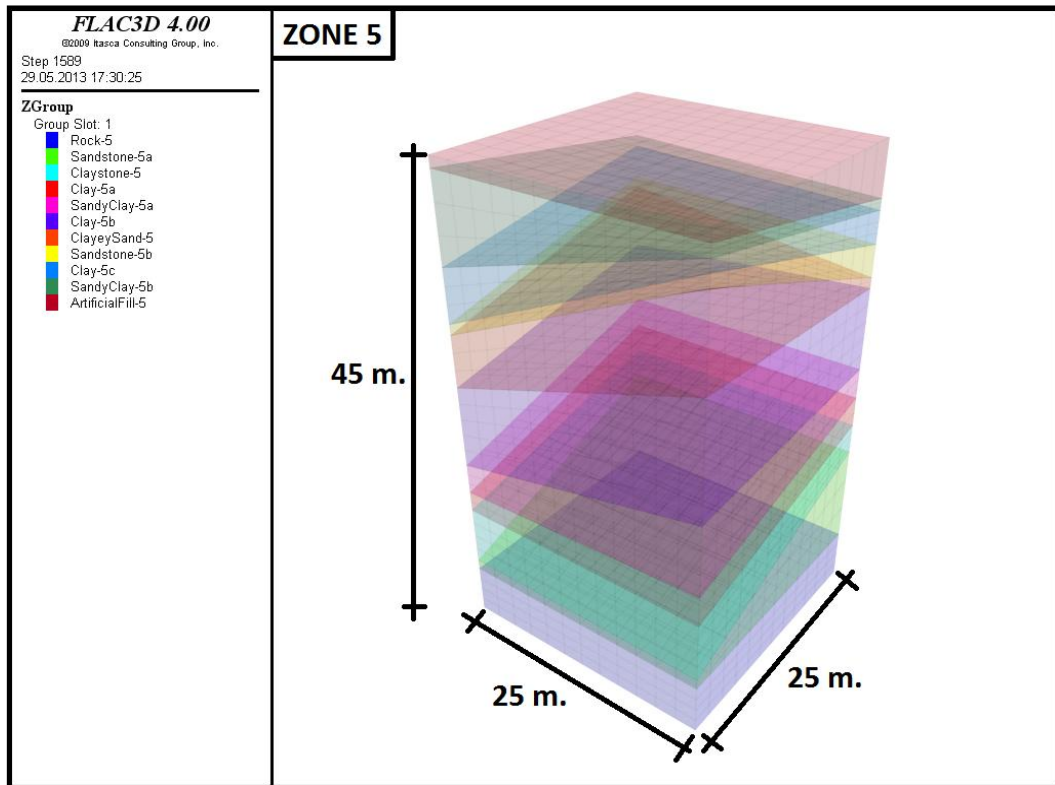


Figure D.5 : Soil property groups for zone 5.

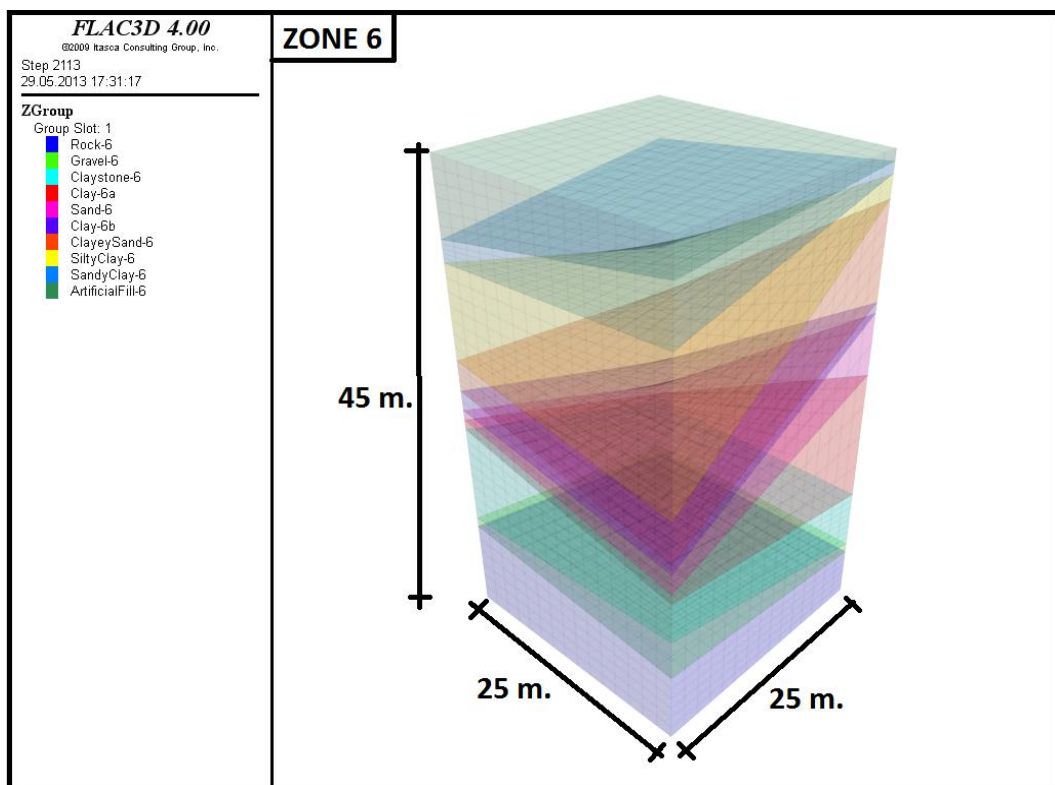


Figure D.6 : Soil property groups for zone 6.

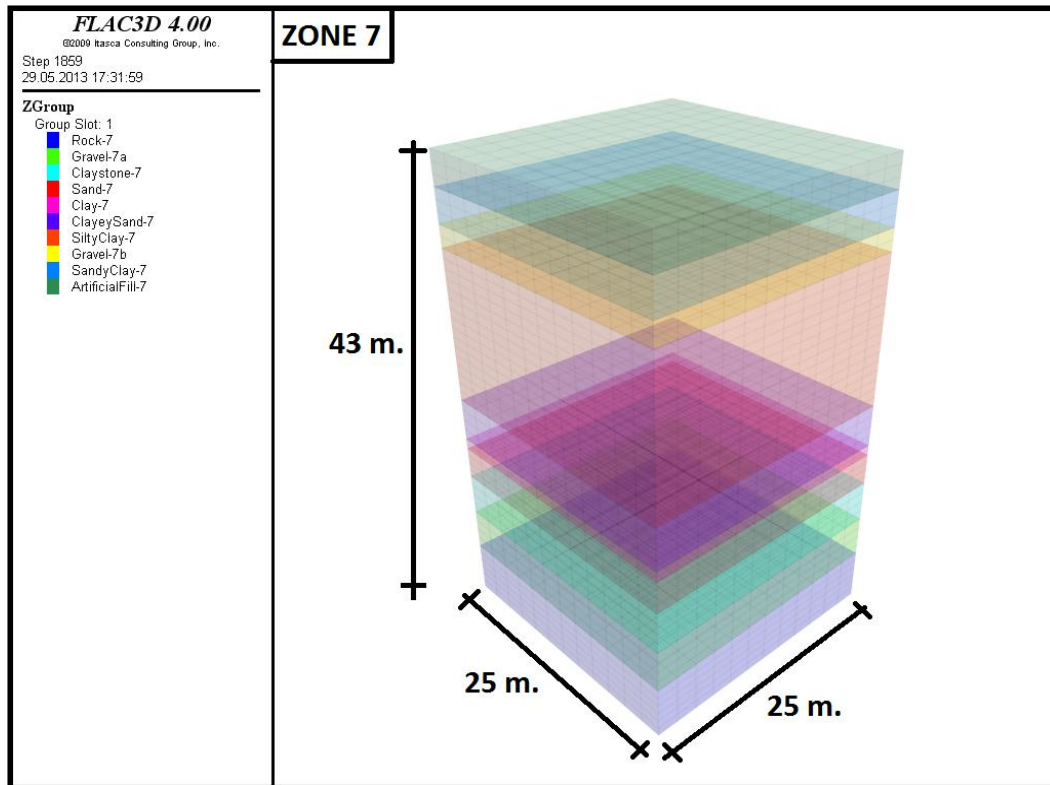


Figure D.7 : Soil property groups for zone 7.

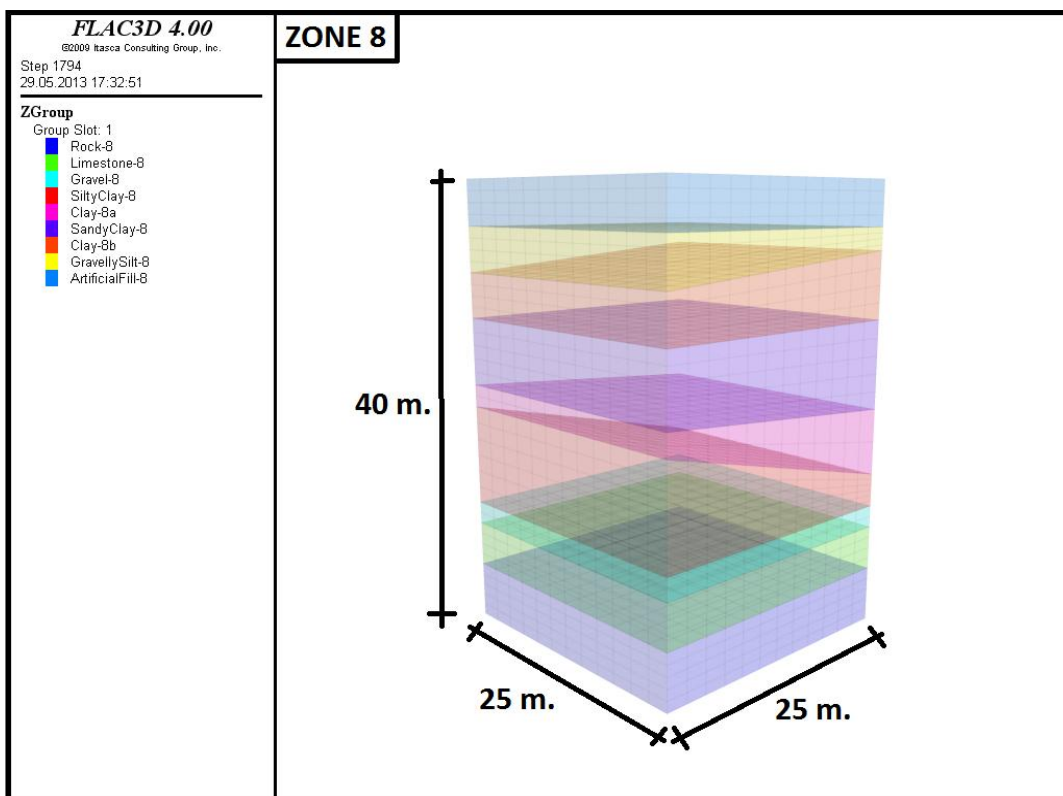


Figure D.8 : Soil property groups for zone 8.

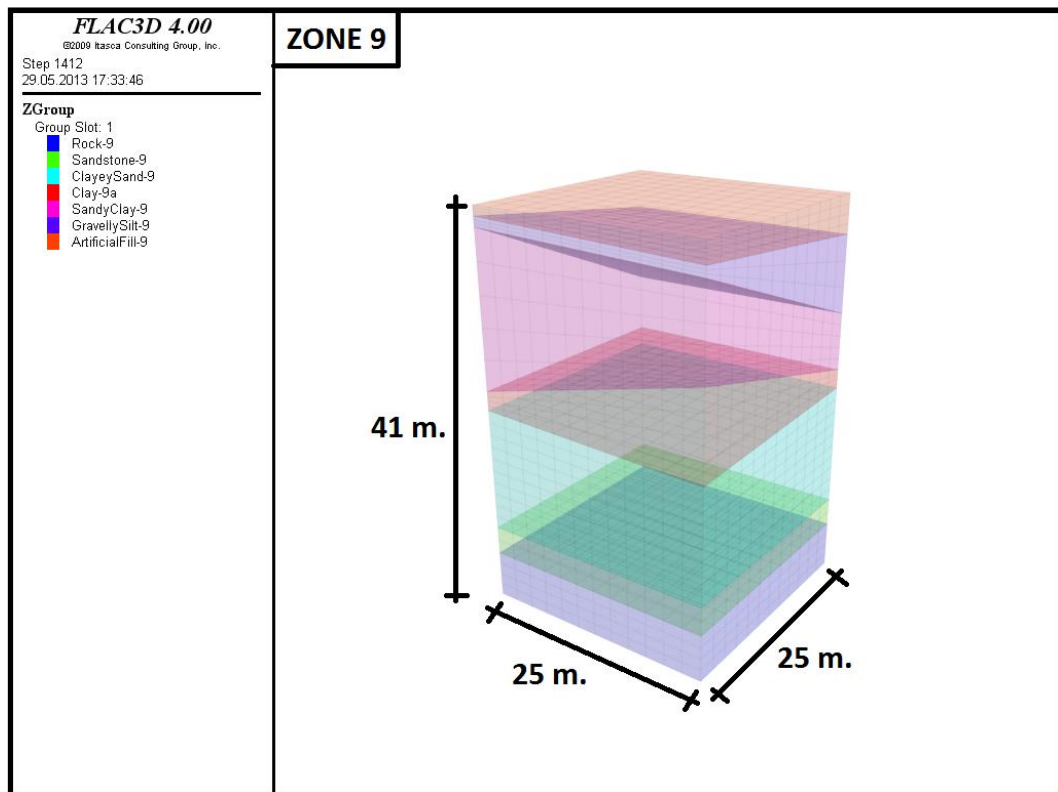


Figure D.9 : Soil property groups for zone 9.

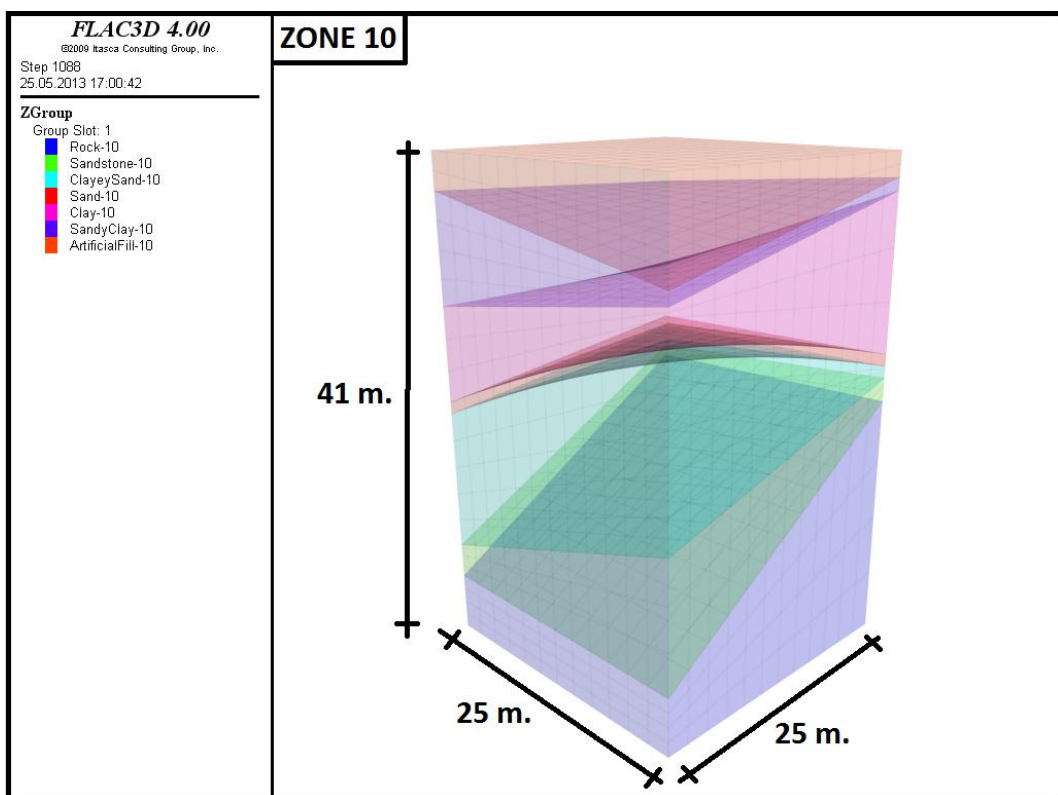


Figure D.10 : Soil property groups for zone 10.

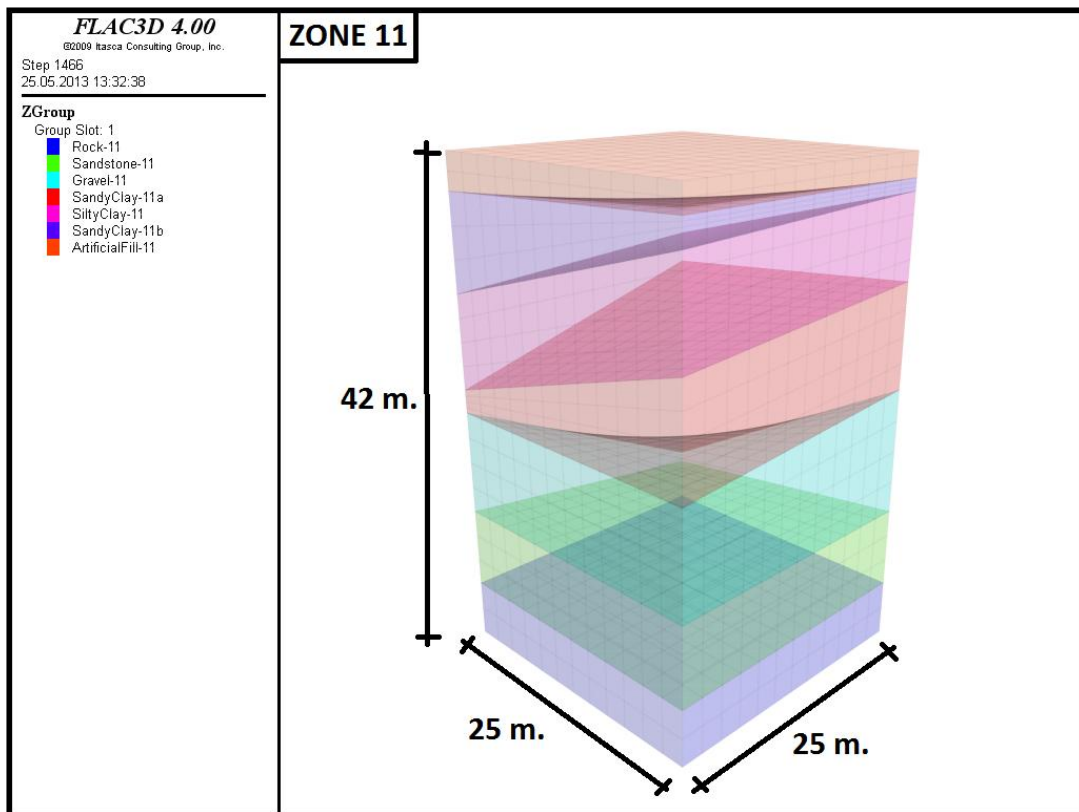


Figure D.11 : Soil property groups for zone 11.

APPENDIX E : Evaluated pore pressure ratio's for all zones.

Table E.1 : Evaluated pore pressure ratio's for all zones.

		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	L 10	L 11		
Depth from bottom		52	48	45	33	30	23	14	12	8	6	2		
Point -1	ZONE 1	1.2	1	0.94	1.08	1.12	0.85	0.72	0.62	0.61	0.63	0.51		
Point -2		1.2	0.81	0.72	0.64	0.79	0.89	0.78	0.62	0.56	0.61	0.69		
Point -3		0.71	0.74	0.92	1.21	0.89	0.98	0.7	0.7	0.65	0.71	0.56		
Point -4		0.72	1.18	0.97	0.92	0.9	0.77	0.63	0.63	0.63	0.71	0.56		
Point -5		1.26	0.99	0.98	0.78	0.76	0.74	0.64	0.62	0.54	0.6	0.49		
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	L 10	L 11	L 12	L 13
Depth from bottom		54	53	52	45	40	37	31	25	21	16	12	7	2
Point -1	ZONE 2	0.78	0.93	1.2	0.95	0.85	0.78	0.71	0.68	0.65	0.59	0.54	0.51	0.49
Point -2		1.15	1.19	1.08	0.81	0.73	0.72	0.69	0.71	0.66	0.66	0.63	0.53	0.51
Point -3		0.89	1	0.92	0.93	0.89	0.87	0.83	0.79	0.72	0.67	0.66	0.53	0.51
Point -4		0.94	0.95	0.85	0.81	0.81	0.79	0.75	0.77	0.67	0.66	0.65	0.53	0.51
Point -5		1.1	0.9	0.87	0.87	0.72	0.71	0.68	0.67	0.7	0.67	0.65	0.6	0.51
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8					
Depth from bottom		44	40	30	22	16	13	7	2					
Point -1	ZONE 3	0.72	0.95	0.91	0.88	0.82	0.88	0.75	0.66					
Point -2		0.84	1.29	1.06	0.93	0.82	0.81	0.75	0.66					
Point -3		0.7	1.01	1.04	0.93	0.81	0.72	0.67	0.6					
Point -4		0.71	0.91	0.81	0.75	0.68	0.65	0.63	0.63					
Point -5		0.7	1.06	0.88	0.81	0.81	0.72	0.63	0.6					
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9				
Depth from bottom		43	41	38	30	25	18	12	7	2				
Point -1	ZONE 4	0.87	1.17	0.86	0.74	0.72	0.71	0.73	0.69	0.62				
Point -2		0.82	1.23	0.93	0.77	0.6	0.68	0.67	0.65	0.58				
Point -3		0.87	1.24	0.91	0.78	0.74	0.72	0.71	0.65	0.61				
Point -4		0.75	1.01	0.84	0.78	0.68	0.69	0.64	0.62	0.62				
Point -5		0.94	0.97	0.84	0.78	0.72	0.68	0.73	0.69	0.59				
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	L 10	L 11		
Depth from bottom		44	42	37	35	32	25	20	15	10	5	2		
Point -1	ZONE 5	0.96	1.28	0.82	0.8	0.73	0.72	0.71	0.76	0.68	0.57	0.48		
Point -2		0.86	1.03	0.9	0.8	0.71	0.65	0.66	0.62	0.66	0.6	0.55		
Point -3		0.79	0.98	0.9	0.85	0.71	0.65	0.63	0.6	0.64	0.6	0.53		
Point -4		0.85	1.08	0.91	0.88	0.65	0.63	0.61	0.59	0.61	0.59	0.55		
Point -5		0.96	0.9	1.01	0.82	0.76	0.79	0.71	0.71	0.67	0.53	0.54		
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	L 10			
Depth from bottom		43	41	35	27	21	18	15	12	7	4			
Point -1	ZONE 6	0.94	1.13	1.03	0.92	0.82	0.79	0.78	0.76	0.5	0.57			
Point -2		1.09	1.02	1.02	0.97	0.8	0.76	0.65	0.62	0.64	0.59			
Point -3		0.99	0.98	0.95	0.82	0.78	0.71	0.68	0.65	0.64	0.53			
Point -4		0.94	1.33	1.01	0.88	0.85	0.86	0.81	0.77	0.66	0.59			
Point -5		1.05	1.06	0.96	0.89	0.87	0.85	0.78	0.74	0.66	0.62			
		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	L 10			
Depth from bottom		42	39	36	27	19	16	14	11	7	2			
Point -1	ZONE 7	0.93	1.21	0.91	0.87	0.76	0.7	0.68	0.64	0.58	0.56			
Point -2		0.89	0.96	0.86	0.84	0.8	0.78	0.7	0.69	0.65	0.58			
Point -3		1.02	1.11	0.82	0.77	0.71	0.63	0.62	0.58	0.55	0.54			
Point -4		1.07	1.08	0.92	0.86	0.79	0.77	0.73	0.6	0.57	0.53			
Point -5		1.01	0.94	0.88	0.85	0.83	0.76	0.65	0.6	0.56	0.51			

Table E.1 (continued) : Evaluated pore pressure ratio's for all zones.

		L 1	L 2	L 3	L 4	L 5	L 6	L 7	L 8	L 9	
Depth from bottom		38	34	30	24	21	15	10	7	2	
Point -1	ZONE 8	0.82	0.91	0.9	0.83	0.84	0.8	0.73	0.58	0.52	
Point -2		0.89	1.05	0.92	0.88	0.85	0.72	0.66	0.52	0.5	
Point -3		0.91	1.02	0.96	0.88	0.81	0.81	0.79	0.57	0.53	
Point -4		0.87	0.96	0.91	0.86	0.83	0.82	0.77	0.53	0.52	
Point -5		0.88	0.94	0.93	0.83	0.8	0.76	0.75	0.53	0.51	
		L 1	L 2	L 3	L 4	L 5	L 6	L 7			
Depth from bottom		40	36	28	22	15	7	2			
Point -1	ZONE 9	0.91	0.86	0.76	0.73	0.7	0.54	0.51			
Point -2		0.87	0.86	0.71	0.7	0.67	0.58	0.57			
Point -3		0.85	0.85	0.82	0.75	0.73	0.58	0.56			
Point -4		0.89	0.88	0.81	0.74	0.68	0.59	0.57			
Point -5		0.95	0.99	0.83	0.78	0.65	0.61	0.59			
		L 1	L 2	L 3	L 4	L 5	L 6	L 7			
Depth from bottom		39	34	29	26	18	9	2			
Point -1	ZONE 10	0.79	0.82	0.8	1.04	0.76	0.71	0.55			
Point -2		0.55	0.94	0.78	0.88	0.61	0.58	0.53			
Point -3		0.5	0.96	0.8	0.86	0.75	0.58	0.41			
Point -4		0.86	0.72	0.86	1.01	0.82	0.68	0.39			
Point -5		0.92	0.96	0.82	0.86	0.72	0.44	0.42			
		L 1	L 2	L 3	L 4	L 5	L 6	L 7			
Depth from bottom		40	35	28	18	12	8	2			
Point -1	ZONE 11	0.49	0.88	0.85	0.74	0.7	0.69	0.44			
Point -2		0.57	0.87	0.74	0.84	0.77	0.61	0.47			
Point -3		0.8	0.93	0.79	0.86	0.75	0.7	0.48			
Point -4		0.68	0.95	0.95	0.94	0.77	0.7	0.47			
Point -5		0.87	0.94	0.82	0.88	0.62	0.48	0.4			

APPENDIX F : Evaluated pore pressure ratio – depth graphs for all zones.

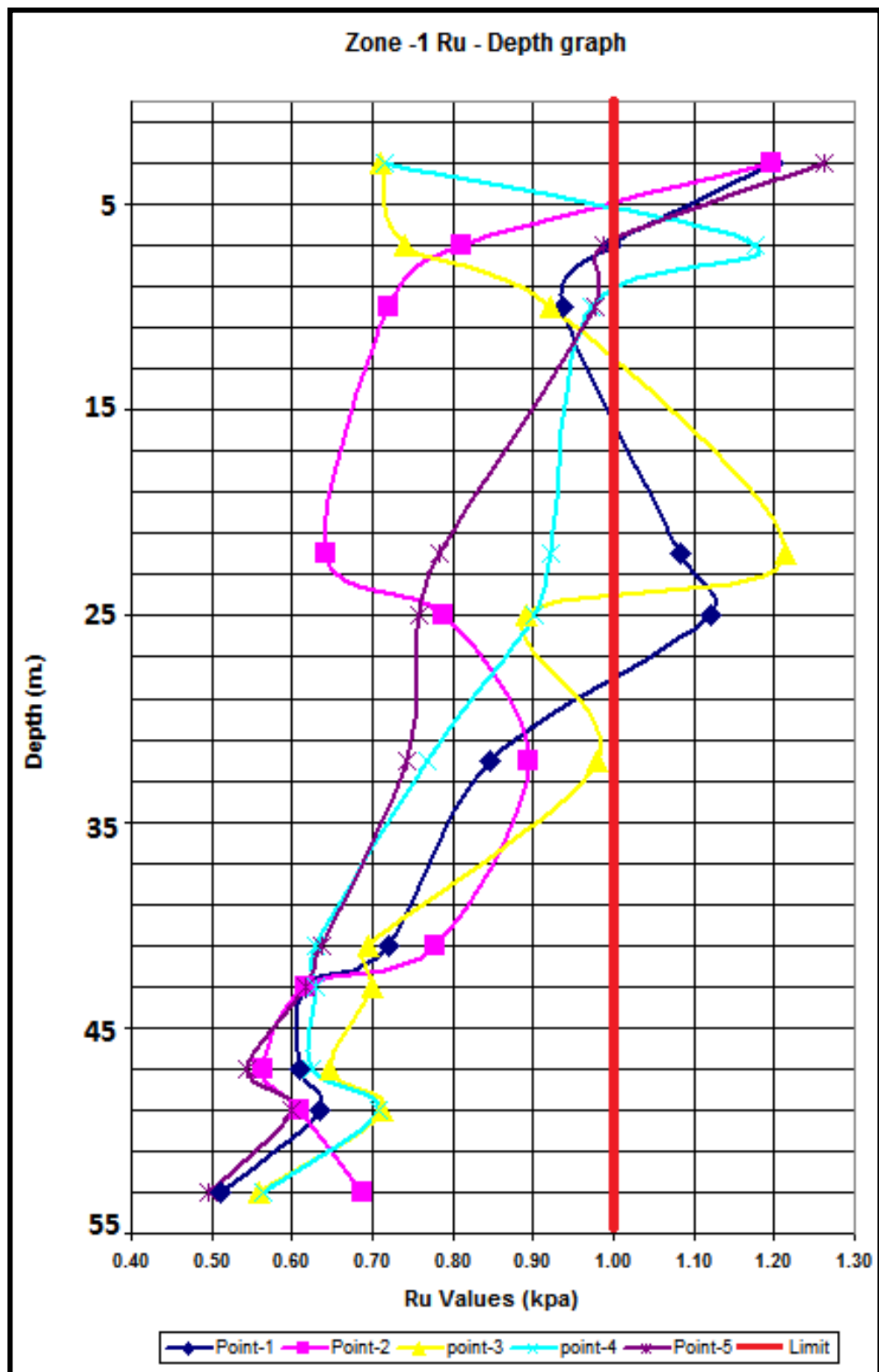


Figure F.1 : Evaluated pore pressure ratio – depth graph of zone 1.

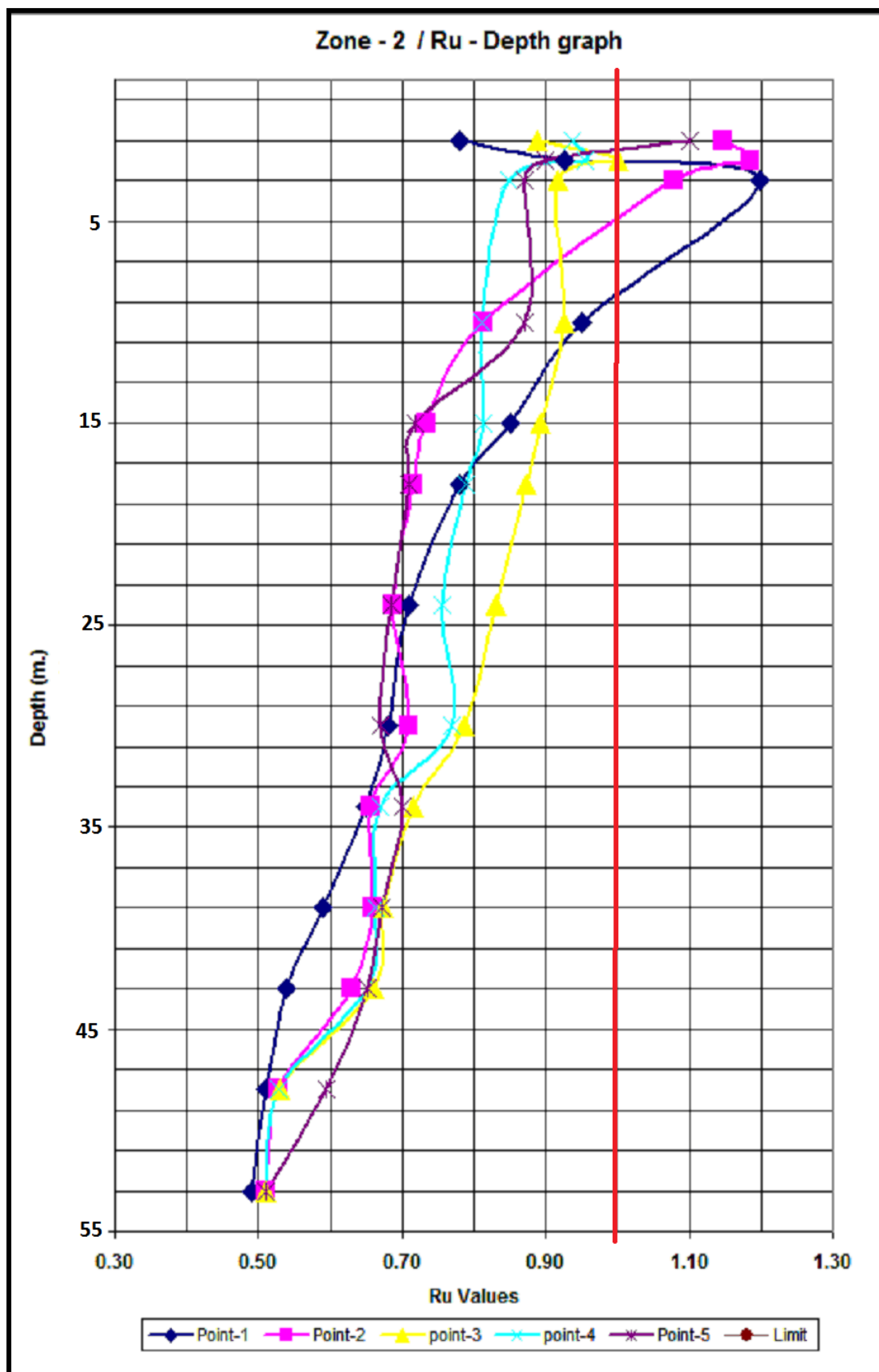


Figure F.2 : Evaluated pore pressure ratio – depth graph of zone 2.

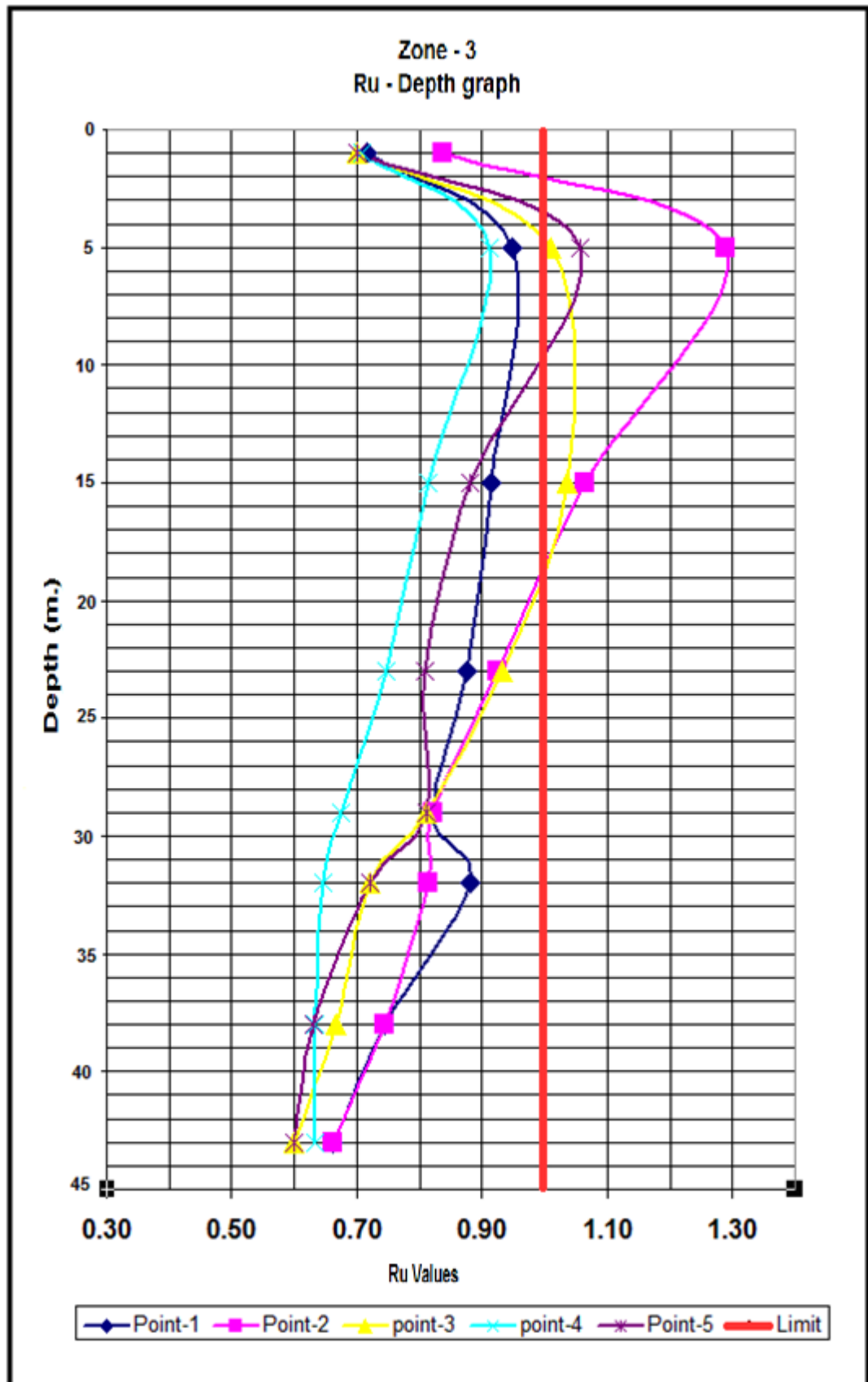


Figure F.3 : Evaluated pore pressure ratio – depth graph of zone 3.

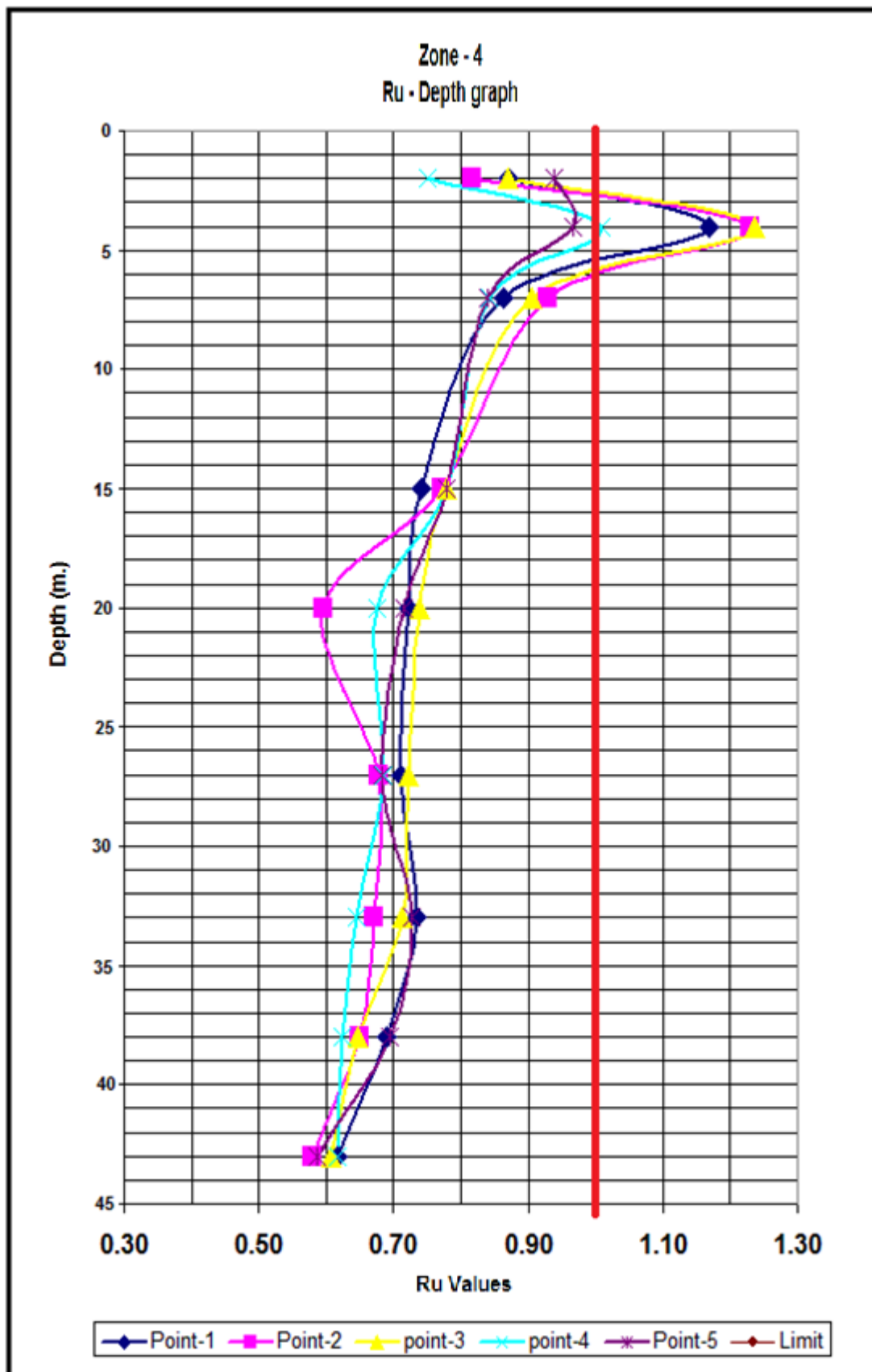


Figure F.4 : Evaluated pore pressure ratio – depth graph of zone 4.

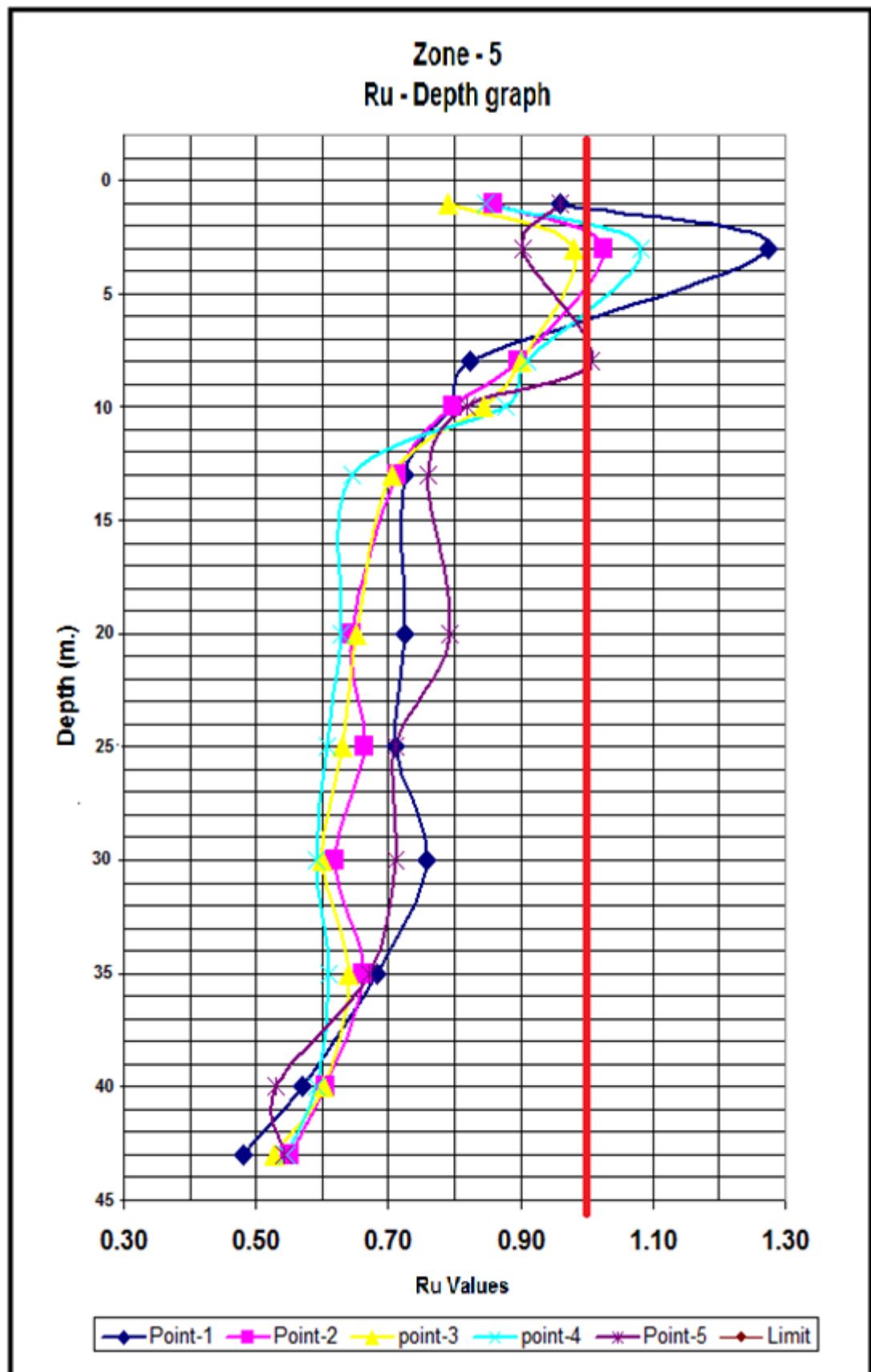


Figure F.5 : Evaluated pore pressure ratio – depth graph of zone 5.

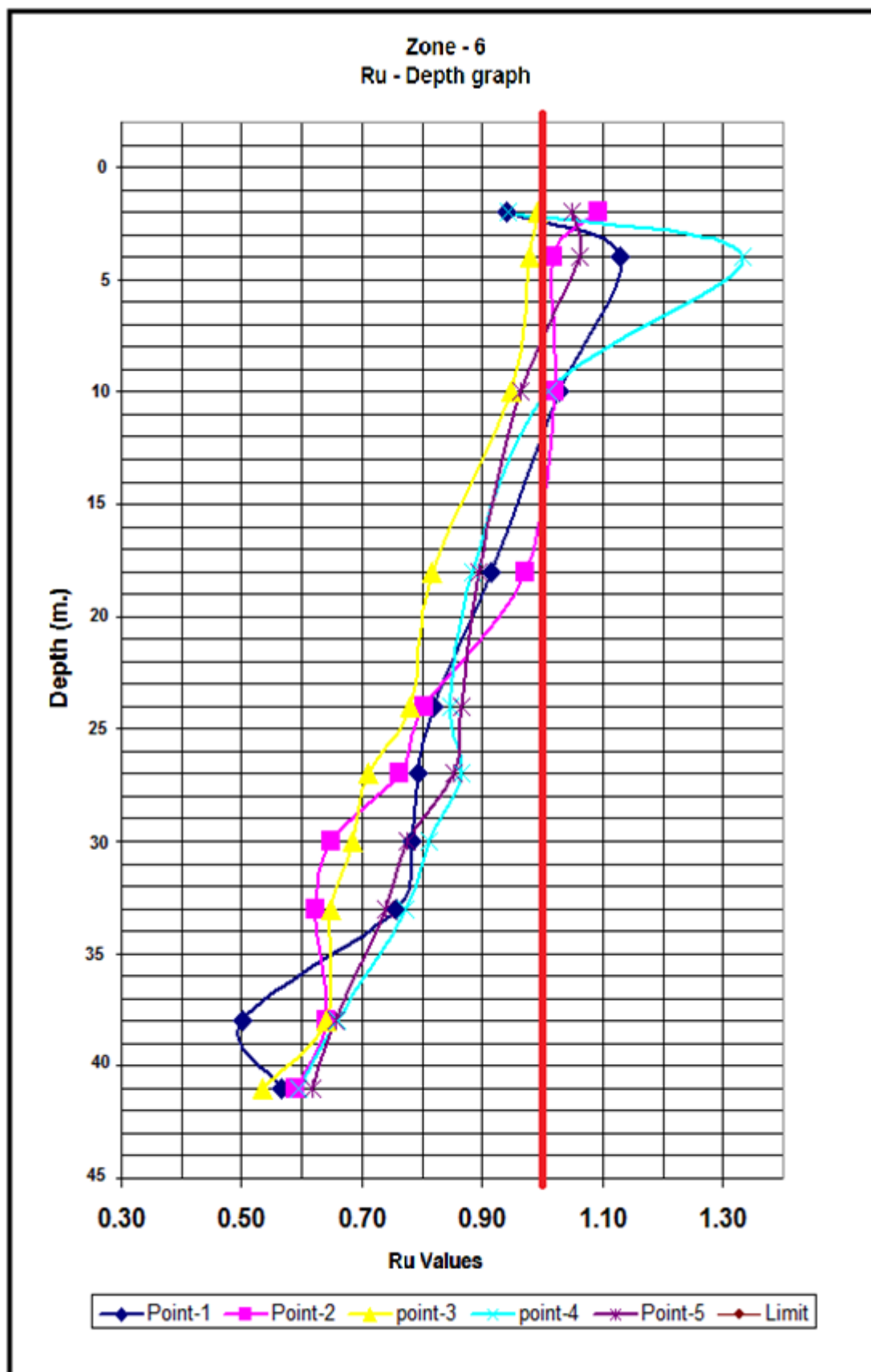


Figure F.6 : Evaluated pore pressure ratio – depth graph of zone 6.

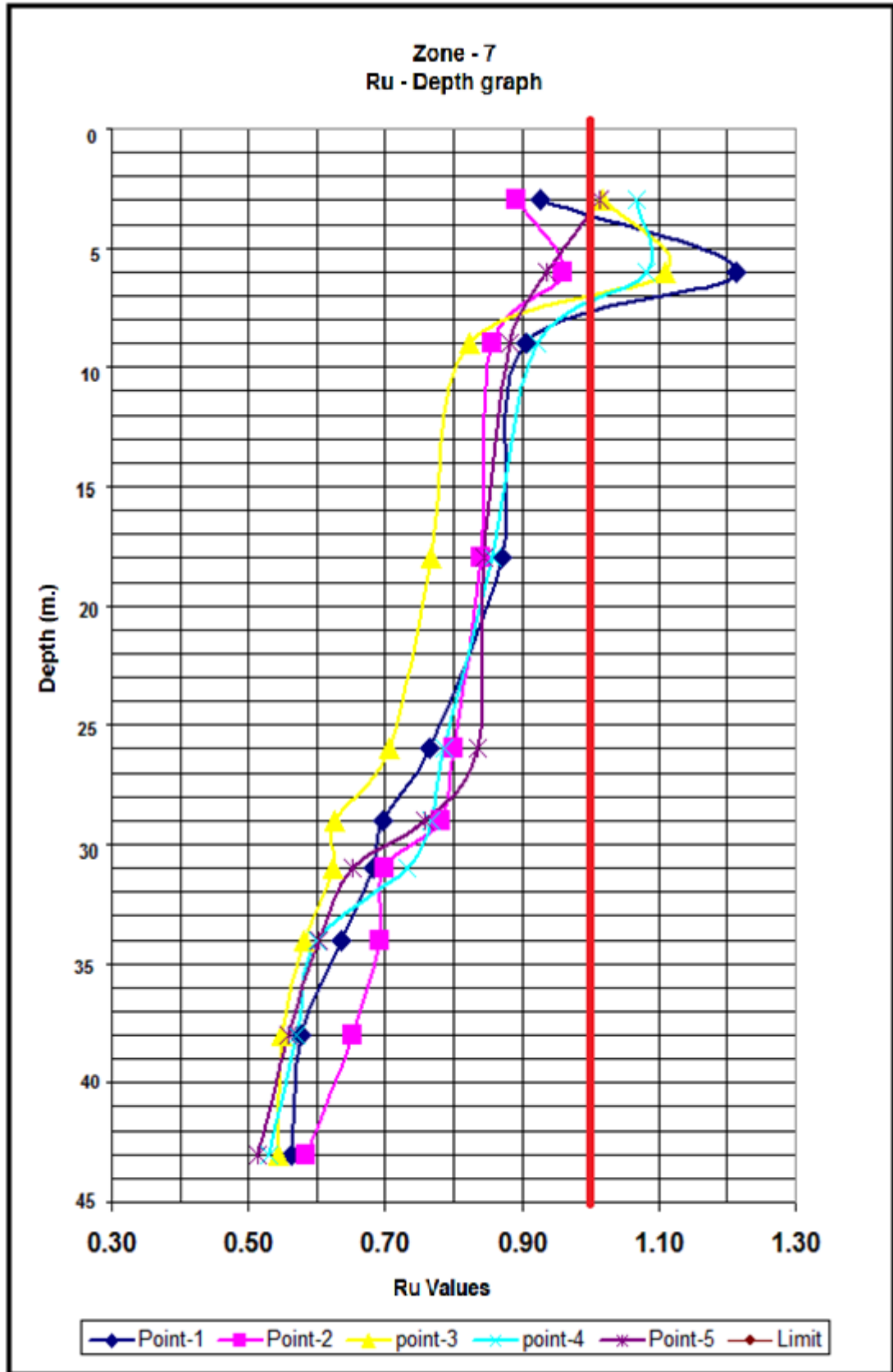


Figure F.7 : Evaluated pore pressure ratio – depth graph of zone 7.

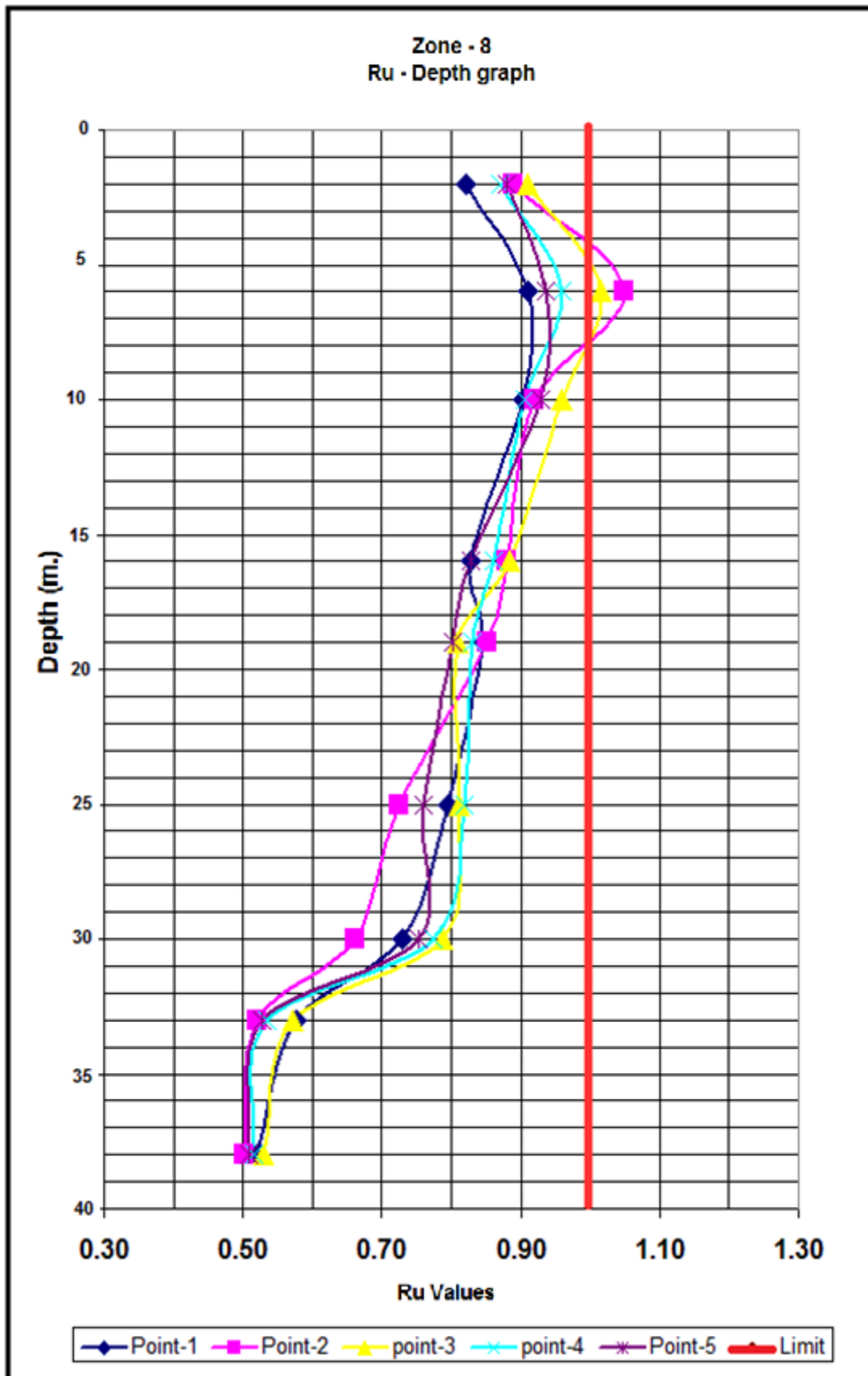


Figure F.8 : Evaluated pore pressure ratio – depth graph of zone 8.

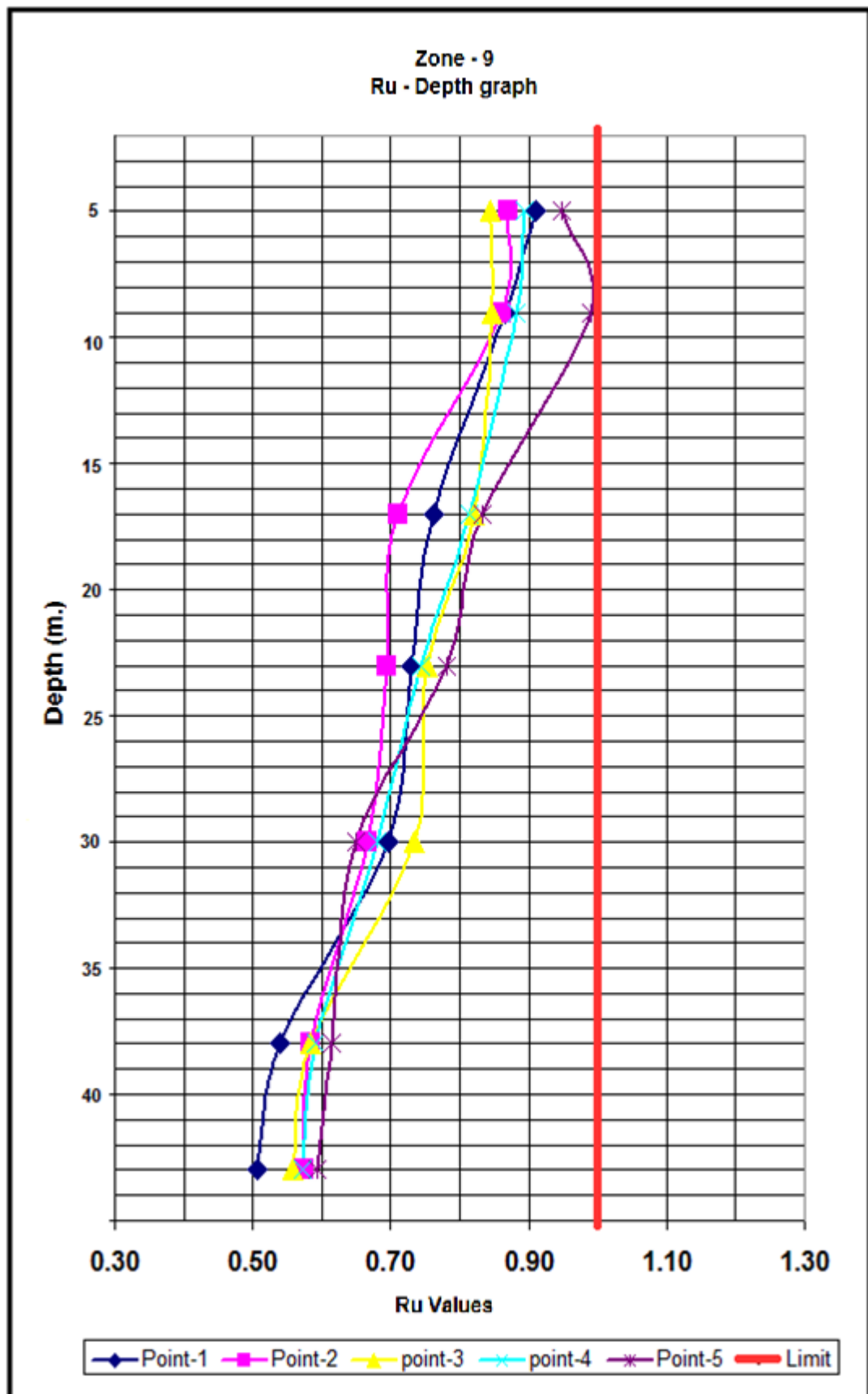


Figure F.9 : Evaluated pore pressure ratio – depth graph of zone 9.

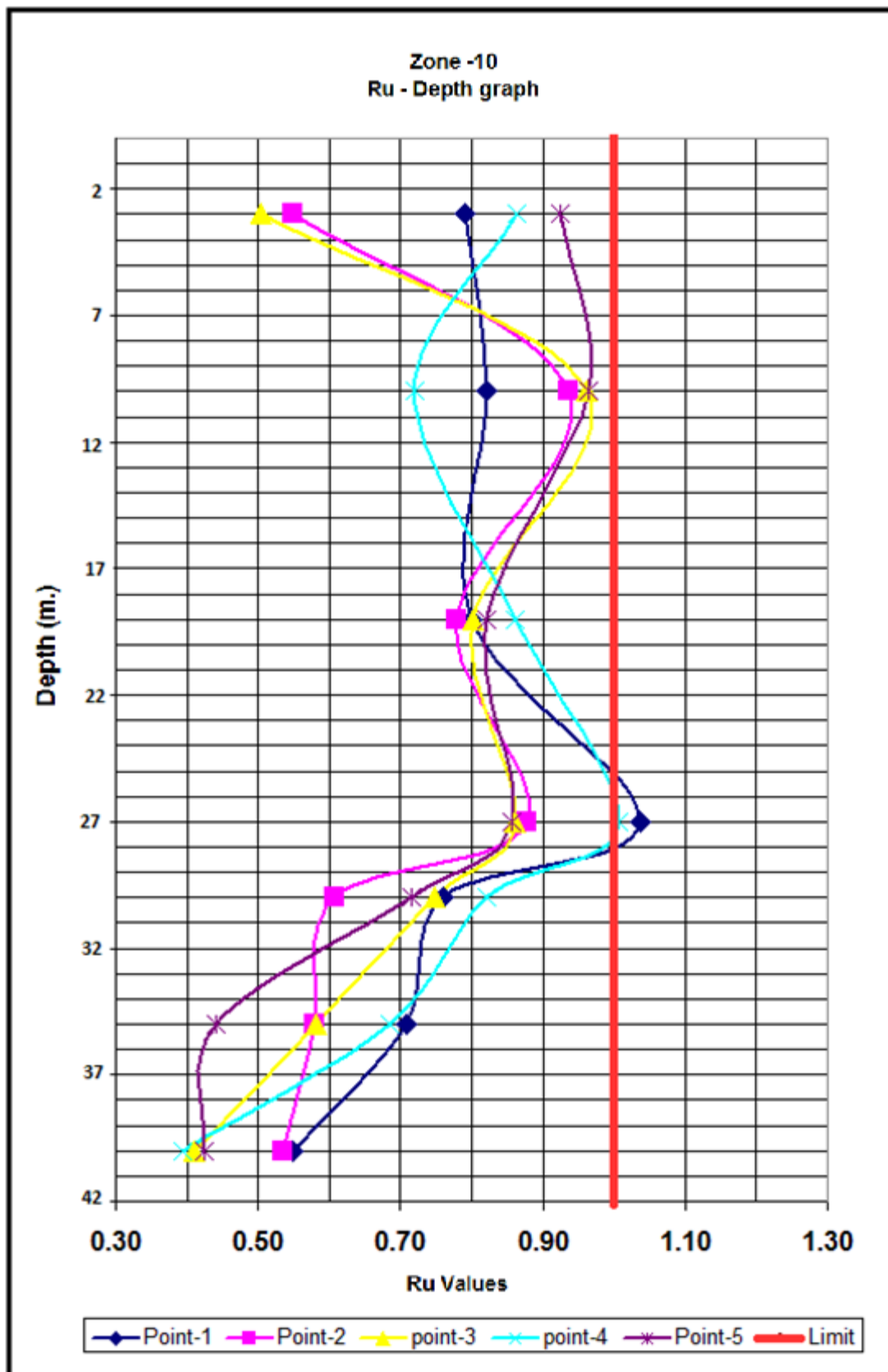


Figure F.10 : Evaluated pore pressure ratio – depth graph of zone 10.

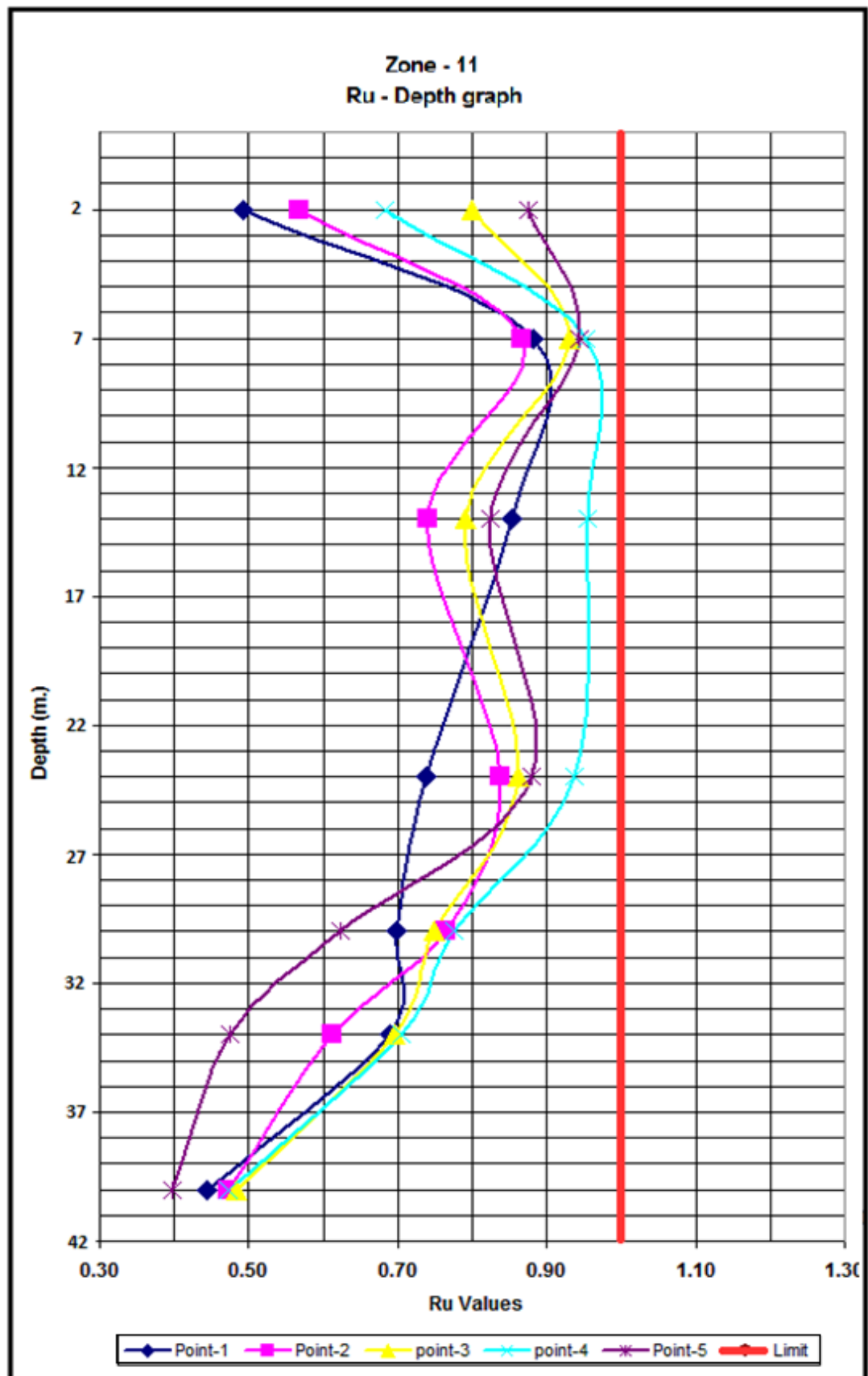


Figure F.11 : Evaluated pore pressure ratio – depth graph of zone 11.

APPENDIX G : Input parameters for neural network models.

Table G.1 : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
37	6	3000000	5000000	1700	20	20000	2.0	1.20
8	2	50000000	100000000	1800	35	0	2.0	1.00
15	4	15000000	33000000	1950	30	0	2.0	0.94
10	10	40000000	80000000	1850	28	20000	2.0	1.08
2	6	40000000	80000000	1850	30	15000	2.0	1.12
3	11	30000000	60000000	1900	22	30000	2.0	0.85
18	3	50000000	100000000	1800	35	0	2.0	0.72
18	3	20000000	40000000	2000	25	20000	2.0	0.62
17	3	40000000	80000000	2200	35	50000	2.0	0.61
17	2	80000000	160000000	2200	40	0	2.0	0.63
24	5	500000000	1000000000	2800	42	10000000	2.0	0.51
37	2	3000000	5000000	1700	20	20000	2.0	1.20
8	2	50000000	100000000	1800	35	0	2.0	0.81
15	1	15000000	33000000	1950	30	0	2.0	0.72
10	16	40000000	80000000	1850	28	20000	2.0	0.64
2	6	40000000	80000000	1850	30	15000	2.0	0.79
3	6	30000000	60000000	1900	22	30000	2.0	0.89
18	3	50000000	100000000	1800	35	0	2.0	0.78
18	6	20000000	40000000	2000	25	20000	2.0	0.62
17	6	40000000	80000000	2200	35	50000	2.0	0.56
17	2	80000000	160000000	2200	40	0	2.0	0.61
24	5	500000000	1000000000	2800	42	10000000	2.0	0.69
37	8	3000000	5000000	1700	20	20000	2.0	0.71
8	2	50000000	100000000	1800	35	0	2.0	0.74
15	3	15000000	33000000	1950	30	0	2.0	0.92
10	13	40000000	80000000	1850	28	20000	2.0	1.21
2	6	40000000	80000000	1850	30	15000	2.0	0.89
3	4	30000000	60000000	1900	22	30000	2.0	0.98
18	6	50000000	100000000	1800	35	0	2.0	0.70
18	3	20000000	40000000	2000	25	20000	2.0	0.70
17	4	40000000	80000000	2200	35	50000	2.0	0.65
17	1	80000000	160000000	2200	40	0	2.0	0.71
24	5	500000000	1000000000	2800	42	10000000	2.0	0.56
37	4	3000000	5000000	1700	20	20000	2.0	0.72
8	7	50000000	100000000	1800	35	0	2.0	1.18
15	1	15000000	33000000	1950	30	0	2.0	0.97
10	9	40000000	80000000	1850	28	20000	2.0	0.92
2	6	40000000	80000000	1850	30	15000	2.0	0.90
3	6	30000000	60000000	1900	22	30000	2.0	0.77
18	3	50000000	100000000	1800	35	0	2.0	0.63
18	6	20000000	40000000	2000	25	20000	2.0	0.63
17	6	40000000	80000000	2200	35	50000	2.0	0.63
17	2	80000000	160000000	2200	40	0	2.0	0.71
24	5	500000000	1000000000	2800	42	10000000	2.0	0.56
37	6	3000000	5000000	1700	20	20000	2.0	1.26
8	2	50000000	100000000	1800	35	0	2.0	0.99

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
15	4	15000000	33000000	1950	30	0	2.0	0.98
10	10	40000000	80000000	1850	28	20000	2.0	0.78
2	6	40000000	80000000	1850	30	15000	2.0	0.76
3	11	30000000	60000000	1900	22	30000	2.0	0.74
18	3	50000000	100000000	1800	35	0	2.0	0.64
18	3	20000000	40000000	2000	25	20000	2.0	0.62
17	3	40000000	80000000	2200	35	50000	2.0	0.54
17	2	80000000	160000000	2200	40	0	2.0	0.60
24	5	500000000	1000000000	2800	42	10000000	2.0	0.49
10	1	3000000	5000000	1700	20	10000	1.0	0.78
5	1	16000000	35000000	1850	26	10000	1.0	0.93
4	1	15000000	33000000	1800	30	0	1.0	1.20
34	11	100000000	200000000	2000	40	1000000	1.0	0.95
13	1	30000000	60000000	1900	22	30000	1.0	0.85
16	6	50000000	100000000	1850	28	20000	1.0	0.78
7	6	25000000	50000000	1850	30	10000	1.0	0.71
9	6	20000000	40000000	1900	22	10000	1.0	0.68
20	3	60000000	120000000	1800	36	0	1.0	0.65
18	6	20000000	40000000	2000	25	20000	1.0	0.59
18	3	35000000	70000000	1900	22	30000	1.0	0.540
17	5	80000000	160000000	2000	40	1000000	1.0	0.51
24	5	500000000	1000000000	2800	42	10000000	1.0	0.49
10	1	3000000	5000000	1700	20	10000	1.0	1.15
5	1	16000000	35000000	1850	26	10000	1.0	1.19
4	3	15000000	33000000	1800	30	0	1.0	1.08
34	1	100000000	200000000	2000	40	1000000	1.0	0.81
13	9	30000000	60000000	1900	22	30000	1.0	0.73
16	5	50000000	100000000	1850	28	20000	1.0	0.72
7	7	25000000	50000000	1850	30	10000	1.0	0.69
9	6	20000000	40000000	1900	22	10000	1.0	0.71
20	3	60000000	120000000	1800	36	0	1.0	0.66
18	6	20000000	40000000	2000	25	20000	1.0	0.66
18	3	35000000	70000000	1900	22	30000	1.0	0.631
17	5	80000000	160000000	2000	40	1000000	1.0	0.53
24	5	500000000	1000000000	2800	42	10000000	1.0	0.51
10	4	3000000	5000000	1700	20	10000	1.0	0.89
5	2	16000000	35000000	1850	26	10000	1.0	1.00
4	6	15000000	33000000	1800	30	0	1.0	0.92
34	4	100000000	200000000	2000	40	1000000	1.0	0.93
13	3	30000000	60000000	1900	22	30000	1.0	0.89
16	1	50000000	100000000	1850	28	20000	1.0	0.87
7	7	25000000	50000000	1850	30	10000	1.0	0.83
9	6	20000000	40000000	1900	22	10000	1.0	0.79
20	3	60000000	120000000	1800	36	0	1.0	0.72
18	6	20000000	40000000	2000	25	20000	1.0	0.67
18	3	35000000	70000000	1900	22	30000	1.0	0.66
17	5	80000000	160000000	2000	40	1000000	1.0	0.53

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
24	5	500000000	1000000000	2800	42	10000000	1.0	0.51
10	3	3000000	5000000	1700	20	10000	1.0	0.94
5	3	16000000	35000000	1850	26	10000	1.0	0.95
4	6	15000000	33000000	1800	30	0	1.0	0.85
34	3	100000000	200000000	2000	40	1000000	1.0	0.81
13	4	30000000	60000000	1900	22	30000	1.0	0.81
16	1	50000000	100000000	1850	28	20000	1.0	0.79
7	7	25000000	50000000	1850	30	10000	1.0	0.75
9	6	20000000	40000000	1900	22	10000	1.0	0.77
20	3	60000000	120000000	1800	36	0	1.0	0.67
18	6	20000000	40000000	2000	25	20000	1.0	0.66
18	3	35000000	70000000	1900	22	30000	1.0	0.65
17	5	80000000	160000000	2000	40	1000000	1.0	0.53
24	5	500000000	1000000000	2800	42	10000000	1.0	0.51
10	2	3000000	5000000	1700	20	10000	1.0	1.10
5	3	16000000	35000000	1850	26	10000	1.0	0.90
4	1	15000000	33000000	1800	30	0	1.0	0.87
34	6	100000000	200000000	2000	40	1000000	1.0	0.87
13	3	30000000	60000000	1900	22	30000	1.0	0.72
16	6	50000000	100000000	1850	28	20000	1.0	0.71
7	6	25000000	50000000	1850	30	10000	1.0	0.68
9	6	20000000	40000000	1900	22	10000	1.0	0.67
20	3	60000000	120000000	1800	36	0	1.0	0.70
18	6	20000000	40000000	2000	25	20000	1.0	0.67
18	3	35000000	70000000	1900	22	30000	1.0	0.65
17	5	80000000	160000000	2000	40	1000000	1.0	0.60
24	5	500000000	1000000000	2800	42	10000000	1.0	0.51
13	2	3000000	5000000	1700	20	10000	2.0	0.72
4	4	13000000	25000000	1850	25	4000	2.0	0.95
2	14	20000000	40000000	1850	25	7000	2.0	0.91
9	7	10000000	20000000	1900	20	10000	2.0	0.88
8	4	15000000	30000000	1850	26	5000	2.0	0.82
8	2	10000000	20000000	1900	20	10000	2.0	0.88
7	7	20000000	40000000	2100	30	10000	2.0	0.75
24	5	500000000	1000000000	2800	42	10000000	2.0	0.66
13	1	3000000	5000000	1700	20	10000	2.0	0.84
4	10	13000000	25000000	1850	25	4000	2.0	1.29
2	9	20000000	40000000	1850	25	7000	2.0	1.06
9	7	10000000	20000000	1900	20	10000	2.0	0.93
8	4	15000000	30000000	1850	26	5000	2.0	0.82
8	2	10000000	20000000	1900	20	10000	2.0	0.81
7	7	20000000	40000000	2100	30	10000	2.0	0.75
24	5	500000000	1000000000	2800	42	10000000	2.0	0.66
13	3	3000000	5000000	1700	20	10000	2.0	0.70
4	12	13000000	25000000	1850	25	4000	2.0	1.01
2	1	20000000	40000000	1850	25	7000	2.0	1.04
9	6	10000000	20000000	1900	20	10000	2.0	0.93

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
8	2	15000000	30000000	1850	26	5000	2.0	0.81
8	9	10000000	20000000	1900	20	10000	2.0	0.72
7	7	20000000	40000000	2100	30	10000	2.0	0.67
24	5	500000000	1000000000	2800	42	10000000	2.0	0.60
13	3	3000000	5000000	1700	20	10000	2.0	0.71
4	2	13000000	25000000	1850	25	4000	2.0	0.91
2	4	20000000	40000000	1850	25	7000	2.0	0.81
9	7	10000000	20000000	1900	20	10000	2.0	0.75
8	8	15000000	30000000	1850	26	5000	2.0	0.68
8	9	10000000	20000000	1900	20	10000	2.0	0.65
7	7	20000000	40000000	2100	30	10000	2.0	0.63
24	5	500000000	1000000000	2800	42	10000000	2.0	0.63
13	2	3000000	5000000	1700	20	10000	2.0	0.70
4	4	13000000	25000000	1850	25	4000	2.0	1.06
2	4	20000000	40000000	1850	25	7000	2.0	0.88
9	8	10000000	20000000	1900	20	10000	2.0	0.81
8	3	15000000	30000000	1850	26	5000	2.0	0.81
8	2	10000000	20000000	1900	20	10000	2.0	0.72
7	7	20000000	40000000	2100	30	10000	2.0	0.63
24	5	500000000	1000000000	2800	42	10000000	2.0	0.60
18	1	15000000	33000000	1900	22	15000	1.5	0.87
7	4	60000000	120000000	1850	30	20000	1.5	1.17
9	6	18000000	36000000	1900	22	15000	1.5	0.86
3	8	20000000	40000000	1850	28	10000	1.5	0.74
13	8	30000000	60000000	1900	22	30000	1.5	0.72
4	4	20000000	40000000	1850	28	10000	1.5	0.71
7	2	20000000	40000000	1900	22	15000	1.5	0.73
19	7	30000000	60000000	2100	35	30000	1.5	0.69
24	5	500000000	1000000000	2800	42	10000000	1.5	0.62
18	1	15000000	33000000	1900	22	15000	1.5	0.82
7	4	60000000	120000000	1850	30	20000	1.5	1.23
9	10	18000000	36000000	1900	22	15000	1.5	0.93
3	5	20000000	40000000	1850	28	10000	1.5	0.77
13	7	30000000	60000000	1900	22	30000	1.5	0.60
4	4	20000000	40000000	1850	28	10000	1.5	0.68
7	2	20000000	40000000	1900	22	15000	1.5	0.67
19	7	30000000	60000000	2100	35	30000	1.5	0.65
24	5	500000000	1000000000	2800	42	10000000	1.5	0.58
18	3	15000000	33000000	1900	22	15000	1.5	0.87
7	1	60000000	120000000	1850	30	20000	1.5	1.24
9	5	18000000	36000000	1900	22	15000	1.5	0.91
3	6	20000000	40000000	1850	28	10000	1.5	0.78
13	1	30000000	60000000	1900	22	30000	1.5	0.74
4	8	20000000	40000000	1850	28	10000	1.5	0.72
7	14	20000000	40000000	1900	22	15000	1.5	0.71
19	2	30000000	60000000	2100	35	30000	1.5	0.65
24	5	500000000	1000000000	2800	42	10000000	1.5	0.61

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
18	4	15000000	33000000	1900	22	15000	1.5	0.75
7	5	60000000	120000000	1850	30	20000	1.5	1.01
9	4	18000000	36000000	1900	22	15000	1.5	0.84
3	1	20000000	40000000	1850	28	10000	1.5	0.78
13	6	30000000	60000000	1900	22	30000	1.5	0.68
4	15	20000000	40000000	1850	28	10000	1.5	0.69
7	3	20000000	40000000	1900	22	15000	1.5	0.64
19	2	30000000	60000000	2100	35	30000	1.5	0.62
24	5	500000000	1000000000	2800	42	10000000	1.5	0.62
18	1	15000000	33000000	1900	22	15000	1.5	0.94
7	4	60000000	120000000	1850	30	20000	1.5	0.97
9	6	18000000	36000000	1900	22	15000	1.5	0.84
3	8	20000000	40000000	1850	28	10000	1.5	0.78
13	9	30000000	60000000	1900	22	30000	1.5	0.72
4	3	20000000	40000000	1850	28	10000	1.5	0.68
7	2	20000000	40000000	1900	22	15000	1.5	0.73
19	7	30000000	60000000	2100	35	30000	1.5	0.69
24	5	500000000	1000000000	2800	42	10000000	1.5	0.59
35	1	5000000	10000000	1700	20	15000	1.0	0.96
4	8	13000000	25000000	1850	25	4000	1.0	1.28
3	5	10000000	20000000	1900	22	20000	1.0	0.82
24	1	100000000	200000000	2000	40	1000000	1.0	0.80
23	5	70000000	140000000	1850	30	20000	1.0	0.73
19	8	50000000	100000000	1900	25	40000	1.0	0.72
10	3	50000000	100000000	1850	30	15000	1.0	0.71
4	2	10000000	20000000	1900	22	10000	1.0	0.76
7	6	20000000	40000000	2100	35	50000	1.0	0.68
19	1	100000000	200000000	2000	40	1000000	1.0	0.57
24	5	500000000	1000000000	2800	42	10000000	1.0	0.48
35	1	5000000	10000000	1700	20	15000	1.0	0.86
4	1	13000000	25000000	1850	25	4000	1.0	1.03
3	1	10000000	20000000	1900	22	20000	1.0	0.90
24	1	100000000	200000000	2000	40	1000000	1.0	0.80
23	8	70000000	140000000	1850	30	20000	1.0	0.71
19	11	50000000	100000000	1900	25	40000	1.0	0.65
10	7	50000000	100000000	1850	30	15000	1.0	0.66
4	3	10000000	20000000	1900	22	10000	1.0	0.62
7	6	20000000	40000000	2100	35	50000	1.0	0.66
19	1	100000000	200000000	2000	40	1000000	1.0	0.60
24	5	500000000	1000000000	2800	42	10000000	1.0	0.55
35	4	5000000	10000000	1700	20	15000	1.0	0.79
4	1	13000000	25000000	1850	25	4000	1.0	0.98
3	3	10000000	20000000	1900	22	20000	1.0	0.90
24	1	100000000	200000000	2000	40	1000000	1.0	0.85
23	6	70000000	140000000	1850	30	20000	1.0	0.71
19	6	50000000	100000000	1900	25	40000	1.0	0.65
10	3	50000000	100000000	1850	30	15000	1.0	0.63

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
4	3	10000000	20000000	1900	22	10000	1.0	0.60
7	8	20000000	40000000	2100	35	50000	1.0	0.64
19	10	100000000	200000000	2000	40	1000000	1.0	0.60
24	5	500000000	1000000000	2800	42	10000000	1.0	0.53
35	5	5000000	10000000	1700	20	15000	1.0	0.85
4	1	13000000	25000000	1850	25	4000	1.0	1.08
3	6	10000000	20000000	1900	22	20000	1.0	0.91
24	3	100000000	200000000	2000	40	1000000	1.0	0.88
23	1	70000000	140000000	1850	30	20000	1.0	0.65
19	8	50000000	100000000	1900	25	40000	1.0	0.63
10	3	50000000	100000000	1850	30	15000	1.0	0.61
4	3	10000000	20000000	1900	22	10000	1.0	0.59
7	3	20000000	40000000	2100	35	50000	1.0	0.61
19	10	100000000	200000000	2000	40	1000000	1.0	0.59
24	5	500000000	1000000000	2800	42	10000000	1.0	0.55
35	2	5000000	10000000	1700	20	15000	1.0	0.96
4	3	13000000	25000000	1850	25	4000	1.0	0.90
3	8	10000000	20000000	1900	22	20000	1.0	1.01
24	2	100000000	200000000	2000	40	1000000	1.0	0.82
23	4	70000000	140000000	1850	30	20000	1.0	0.76
19	2	50000000	100000000	1900	25	40000	1.0	0.79
10	7	50000000	100000000	1850	30	15000	1.0	0.71
4	3	10000000	20000000	1900	22	10000	1.0	0.71
7	2	20000000	40000000	2100	35	50000	1.0	0.67
19	7	100000000	200000000	2000	40	1000000	1.0	0.53
24	5	500000000	1000000000	2800	42	10000000	1.0	0.54
2	7	2000000	4000000	1700	20	5000	2.0	0.94
15	2	50000000	100000000	1850	30	15000	2.0	1.13
10	9	15000000	30000000	2000	25	18000	2.0	1.03
21	3	70000000	140000000	1850	30	23000	2.0	0.92
18	2	40000000	80000000	1900	24	30000	2.0	0.82
6	1	20000000	40000000	1800	32	0	2.0	0.79
4	1	10000000	20000000	1900	22	10000	2.0	0.78
14	10	40000000	80000000	2100	35	50000	2.0	0.76
7	5	50000000	100000000	2000	34	0	2.0	0.50
24	5	500000000	1000000000	2800	42	1000000	2.0	0.57
2	5	2000000	4000000	1700	20	5000	2.0	1.09
15	5	50000000	100000000	1850	30	15000	2.0	1.02
10	14	15000000	30000000	2000	25	18000	2.0	1.02
21	4	70000000	140000000	1850	30	23000	2.0	0.97
18	1	40000000	80000000	1900	24	30000	2.0	0.80
6	2	20000000	40000000	1800	32	0	2.0	0.76
4	1	10000000	20000000	1900	22	10000	2.0	0.65
14	4	40000000	80000000	2100	35	50000	2.0	0.62
7	4	50000000	100000000	2000	34	0	2.0	0.64
24	5	500000000	1000000000	2800	42	1000000	2.0	0.59
2	4	2000000	4000000	1700	20	5000	2.0	0.99

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
15	9	50000000	100000000	1850	30	15000	2.0	0.98
10	5	15000000	30000000	2000	25	18000	2.0	0.95
21	8	70000000	140000000	1850	30	23000	2.0	0.82
18	3	40000000	80000000	1900	24	30000	2.0	0.78
6	1	20000000	40000000	1800	32	0	2.0	0.71
4	1	10000000	20000000	1900	22	10000	2.0	0.68
14	5	40000000	80000000	2100	35	50000	2.0	0.65
7	1	50000000	100000000	2000	34	0	2.0	0.64
24	5	500000000	1000000000	2800	42	1000000	2.0	0.53
2	1	2000000	4000000	1700	20	5000	2.0	0.94
15	1	50000000	100000000	1850	30	15000	2.0	1.33
10	2	15000000	30000000	2000	25	18000	2.0	1.01
21	9	70000000	140000000	1850	30	23000	2.0	0.88
18	1	40000000	80000000	1900	24	30000	2.0	0.85
6	6	20000000	40000000	1800	32	0	2.0	0.86
4	12	10000000	20000000	1900	22	10000	2.0	0.81
14	7	40000000	80000000	2100	35	50000	2.0	0.77
7	1	50000000	100000000	2000	34	0	2.0	0.66
24	5	500000000	1000000000	2800	42	1000000	2.0	0.59
2	8	2000000	4000000	1700	20	5000	2.0	1.05
15	1	50000000	100000000	1850	30	15000	2.0	1.06
10	1	15000000	30000000	2000	25	18000	2.0	0.96
21	8	70000000	140000000	1850	30	23000	2.0	0.89
18	3	40000000	80000000	1900	24	30000	2.0	0.87
6	3	20000000	40000000	1800	32	0	2.0	0.85
4	2	10000000	20000000	1900	22	10000	2.0	0.78
14	10	40000000	80000000	2100	35	50000	2.0	0.74
7	2	50000000	100000000	2000	34	0	2.0	0.66
24	5	500000000	1000000000	2800	42	1000000	2.0	0.62
57	3	10000000	20000000	1700	22	20000	1.5	0.93
17	3	50000000	100000000	1850	30	15000	1.5	1.21
2	2	15000000	30000000	2000	25	0	1.5	0.91
8	14	15000000	30000000	2000	24	17000	1.5	0.87
19	4	55000000	110000000	1850	28	25000	1.5	0.76
6	1	10000000	20000000	1900	22	20000	1.5	0.70
5	3	20000000	40000000	1800	30	0	1.5	0.68
4	4	15000000	30000000	2100	32	20000	1.5	0.64
3	4	20000000	40000000	2000	30	0	1.5	0.58
24	5	500000000	1000000000	2800	42	1000000	1.5	0.56
57	3	10000000	20000000	1700	22	20000	1.5	0.89
17	3	50000000	100000000	1850	30	15000	1.5	0.96
2	2	15000000	30000000	2000	25	0	1.5	0.86
8	14	15000000	30000000	2000	24	17000	1.5	0.84
19	4	55000000	110000000	1850	28	25000	1.5	0.80
6	1	10000000	20000000	1900	22	20000	1.5	0.78
5	3	20000000	40000000	1800	30	0	1.5	0.70
4	4	15000000	30000000	2100	32	20000	1.5	0.69

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
3	4	20000000	40000000	2000	30	0	1.5	0.65
24	5	500000000	1000000000	2800	42	1000000	1.5	0.58
57	3	100000000	200000000	1700	22	20000	1.5	1.02
17	3	500000000	1000000000	1850	30	15000	1.5	1.11
2	2	150000000	300000000	2000	25	0	1.5	0.82
8	14	150000000	300000000	2000	24	17000	1.5	0.77
19	4	550000000	1100000000	1850	28	25000	1.5	0.71
6	1	100000000	200000000	1900	22	20000	1.5	0.63
5	3	200000000	400000000	1800	30	0	1.5	0.62
4	4	150000000	300000000	2100	32	20000	1.5	0.58
3	4	200000000	400000000	2000	30	0	1.5	0.55
24	5	500000000	1000000000	2800	42	1000000	1.5	0.54
57	3	100000000	200000000	1700	22	20000	1.5	1.07
17	3	500000000	1000000000	1850	30	15000	1.5	1.08
2	2	150000000	300000000	2000	25	0	1.5	0.92
8	14	150000000	300000000	2000	24	17000	1.5	0.86
19	4	550000000	1100000000	1850	28	25000	1.5	0.79
6	1	100000000	200000000	1900	22	20000	1.5	0.77
5	3	200000000	400000000	1800	30	0	1.5	0.73
4	4	150000000	300000000	2100	32	20000	1.5	0.60
3	4	200000000	400000000	2000	30	0	1.5	0.57
24	5	500000000	1000000000	2800	42	1000000	1.5	0.53
57	3	100000000	200000000	1700	22	20000	1.5	1.01
17	3	500000000	1000000000	1850	30	15000	1.5	0.94
2	2	150000000	300000000	2000	25	0	1.5	0.88
8	14	150000000	300000000	2000	24	17000	1.5	0.85
19	4	550000000	1100000000	1850	28	25000	1.5	0.83
6	1	100000000	200000000	1900	22	20000	1.5	0.76
5	3	200000000	400000000	1800	30	0	1.5	0.65
4	4	150000000	300000000	2100	32	20000	1.5	0.60
3	4	200000000	400000000	2000	30	0	1.5	0.56
24	5	500000000	1000000000	2800	42	1000000	1.5	0.51
20	4	50000000	100000000	1700	22	15000	2.0	0.82
19	4	500000000	1000000000	2050	30	10000	2.0	0.91
2	4	100000000	200000000	1900	22	10000	2.0	0.90
10	6	500000000	1000000000	1850	28	10000	2.0	0.83
12	2	300000000	600000000	1900	22	20000	2.0	0.84
4	9	100000000	200000000	2000	25	10000	2.0	0.80
19	2	1000000000	2000000000	2000	34	0	2.0	0.73
20	4	2000000000	4000000000	2700	35	1000000	2.0	0.58
24	5	1000000000	2000000000	2800	42	1000000	2.0	0.52
20	4	50000000	100000000	1700	22	15000	2.0	0.89
19	4	500000000	1000000000	2050	30	10000	2.0	1.05
2	4	100000000	200000000	1900	22	10000	2.0	0.92
10	6	500000000	1000000000	1850	28	10000	2.0	0.88
12	2	300000000	600000000	1900	22	20000	2.0	0.85
4	9	100000000	200000000	2000	25	10000	2.0	0.72

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
19	2	100000000	200000000	2000	34	0	2.0	0.66
20	4	2000000000	4000000000	2700	35	1000000	2.0	0.52
24	5	1000000000	2000000000	2800	42	1000000	2.0	0.50
20	4	5000000	10000000	1700	22	15000	2.0	0.91
19	2	50000000	100000000	2050	30	10000	2.0	1.02
2	6	10000000	20000000	1900	22	10000	2.0	0.96
10	8	50000000	100000000	1850	28	10000	2.0	0.88
12	6	30000000	60000000	1900	22	20000	2.0	0.81
4	3	10000000	20000000	2000	25	10000	2.0	0.81
19	2	100000000	200000000	2000	34	0	2.0	0.79
20	4	2000000000	4000000000	2700	35	1000000	2.0	0.57
24	5	1000000000	2000000000	2800	42	1000000	2.0	0.53
20	4	5000000	10000000	1700	22	15000	2.0	0.87
19	2	50000000	100000000	2050	30	10000	2.0	0.96
2	6	10000000	20000000	1900	22	10000	2.0	0.91
10	8	50000000	100000000	1850	28	10000	2.0	0.86
12	6	30000000	60000000	1900	22	20000	2.0	0.83
4	3	10000000	20000000	2000	25	10000	2.0	0.82
19	2	100000000	200000000	2000	34	0	2.0	0.77
20	4	2000000000	4000000000	2700	35	1000000	2.0	0.53
24	5	1000000000	2000000000	2800	42	1000000	2.0	0.52
20	4	5000000	10000000	1700	22	15000	2.0	0.88
19	4	50000000	100000000	2050	30	10000	2.0	0.94
2	4	10000000	20000000	1900	22	10000	2.0	0.93
10	6	50000000	100000000	1850	28	10000	2.0	0.83
12	2	30000000	60000000	1900	22	20000	2.0	0.80
4	9	10000000	20000000	2000	25	10000	2.0	0.76
19	2	100000000	200000000	2000	34	0	2.0	0.75
20	4	2000000000	4000000000	2700	35	1000000	2.0	0.53
24	5	1000000000	2000000000	2800	42	1000000	2.0	0.51
15	1	3000000	5000000	1700	20	10000	1.0	0.91
20	1	50000000	100000000	2050	30	10000	1.0	0.86
9	16	50000000	100000000	1850	30	15000	1.0	0.76
12	2	30000000	60000000	1900	22	30000	1.0	0.73
7	13	50000000	100000000	1850	28	20000	1.0	0.70
20	3	80000000	160000000	2000	40	1000000	1.0	0.54
24	5	500000000	1000000000	2800	42	1000000	1.0	0.51
15	1	3000000	5000000	1700	20	10000	1.0	0.87
20	1	50000000	100000000	2050	30	10000	1.0	0.86
9	9	50000000	100000000	1850	30	15000	1.0	0.71
12	9	30000000	60000000	1900	22	30000	1.0	0.70
7	12	50000000	100000000	1850	28	20000	1.0	0.67
20	3	80000000	160000000	2000	40	1000000	1.0	0.58
24	5	500000000	1000000000	2800	42	1000000	1.0	0.57
15	4	3000000	5000000	1700	20	10000	1.0	0.85
20	8	50000000	100000000	2050	30	10000	1.0	0.85
9	6	50000000	100000000	1850	30	15000	1.0	0.82

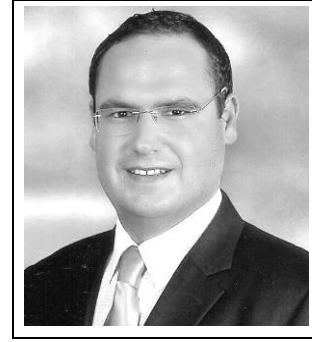
Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
12	2	30000000	60000000	1900	22	30000	1.0	0.75
7	13	50000000	100000000	1850	28	20000	1.0	0.73
20	3	80000000	160000000	2000	40	1000000	1.0	0.58
24	5	500000000	1000000000	2800	42	1000000	1.0	0.56
15	4	3000000	5000000	1700	20	10000	1.0	0.89
20	8	50000000	100000000	2050	30	10000	1.0	0.88
9	6	50000000	100000000	1850	30	15000	1.0	0.81
12	2	30000000	60000000	1900	22	30000	1.0	0.74
7	13	50000000	100000000	1850	28	20000	1.0	0.68
20	3	80000000	160000000	2000	40	1000000	1.0	0.59
24	5	500000000	1000000000	2800	42	1000000	1.0	0.57
15	2	3000000	5000000	1700	20	10000	1.0	0.95
20	6	50000000	100000000	2050	30	10000	1.0	0.99
9	6	50000000	100000000	1850	30	15000	1.0	0.83
12	2	30000000	60000000	1900	22	30000	1.0	0.78
7	13	50000000	100000000	1850	28	20000	1.0	0.65
20	3	80000000	160000000	2000	40	1000000	1.0	0.61
24	5	500000000	1000000000	2800	42	1000000	1.0	
14	3	5000000	10000000	1700	20	10000	2.5	
15	9	40000000	80000000	1850	30	15000	2.5	
10	8	30000000	60000000	1900	22	30000	2.5	
13	1	40000000	80000000	1800	35	0	2.5	
7	12	50000000	100000000	1850	28	20000	2.5	
9	3	80000000	160000000	2000	40	0	2.5	
24	5	10000000000	20000000000	2800	42	1000000	2.5	
14	7	5000000	10000000	1700	20	10000	2.5	
15	1	40000000	80000000	1850	30	15000	2.5	
10	1	30000000	60000000	1900	22	30000	2.5	
13	1	40000000	80000000	1800	35	0	2.5	
7	15	50000000	100000000	1850	28	20000	2.5	
9	11	80000000	160000000	2000	40	0	2.5	
24	5	10000000000	20000000000	2800	42	1000000	2.5	
14	4	5000000	10000000	1700	20	10000	2.5	
15	8	40000000	80000000	1850	30	15000	2.5	
10	5	30000000	60000000	1900	22	30000	2.5	
13	1	40000000	80000000	1800	35	0	2.5	
7	1	50000000	100000000	1850	28	20000	2.5	
9	1	80000000	160000000	2000	40	0	2.5	
24	20	10000000000	20000000000	2800	42	1000000	2.5	
14	2	5000000	10000000	1700	20	10000	2.5	
15	1	40000000	80000000	1850	30	15000	2.5	
10	13	30000000	60000000	1900	22	30000	2.5	
13	1	40000000	80000000	1800	35	0	2.5	
7	1	50000000	100000000	1850	28	20000	2.5	
9	2	80000000	160000000	2000	40	0	2.5	
24	21	10000000000	20000000000	2800	42	1000000	2.5	
14	3	5000000	10000000	1700	20	10000	2.5	

Table G.1 (continued) : Input parameters for neural network models.

SPT - N60	Thickness of Layer (m)	Shear Modulus (Pa)	Bulk Modulus (Pa)	Density (kg/m ³)	Friction Angle (degree)	Cohesion (Pa)	Water Table (m)	r _u
15	9	40000000	80000000	1850	30	15000	2.5	
10	9	30000000	60000000	1900	22	30000	2.5	
13	2	40000000	80000000	1800	35	0	2.5	
7	7	50000000	100000000	1850	28	20000	2.5	
9	3	80000000	160000000	2000	40	0	2.5	
24	5	1000000000	2000000000	2800	42	1000000	2.5	
30	3	5000000	10000000	1700	20	10000	2.0	
9	8	60000000	120000000	1850	30	15000	2.0	
11	8	15000000	32000000	2000	25	15000	2.0	
13	2	45000000	90000000	1850	30	15000	2.0	
10	9	90000000	200000000	2000	34	0	2.0	
19	7	50000000	100000000	2000	40	0	2.0	
24	5	1000000000	2000000000	2800	42	1000000	2.0	
30	2	5000000	10000000	1700	20	10000	2.0	
9	1	60000000	120000000	1850	30	15000	2.0	
11	9	15000000	32000000	2000	25	15000	2.0	
13	9	45000000	90000000	1850	30	15000	2.0	
10	9	90000000	200000000	2000	34	0	2.0	
19	7	50000000	100000000	2000	40	0	2.0	
24	5	1000000000	2000000000	2800	42	1000000	2.0	
30	7	5000000	10000000	1700	20	10000	2.0	
9	4	60000000	120000000	1850	30	15000	2.0	
11	1	15000000	32000000	2000	25	15000	2.0	
13	20	45000000	90000000	1850	30	15000	2.0	
10	1	90000000	200000000	2000	34	0	2.0	
19	4	50000000	100000000	2000	40	0	2.0	
24	5	1000000000	2000000000	2800	42	1000000	2.0	
30	2	5000000	10000000	1700	20	10000	2.0	
9	1	60000000	120000000	1850	30	15000	2.0	
11	7	15000000	32000000	2000	25	15000	2.0	
13	9	45000000	90000000	1850	30	15000	2.0	
10	11	90000000	200000000	2000	34	0	2.0	
19	7	50000000	100000000	2000	40	0	2.0	
24	5	1000000000	2000000000	2800	42	1000000	2.0	
30	3	5000000	10000000	1700	20	10000	2.0	
9	2	60000000	120000000	1850	30	15000	2.0	
11	14	15000000	32000000	2000	25	15000	2.0	
13	2	45000000	90000000	1850	30	15000	2.0	
10	9	90000000	200000000	2000	34	0	2.0	
19	7	50000000	100000000	2000	40	0	2.0	
24	8	1000000000	2000000000	2800	42	1000000	2.0	

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List of Publications and Patents:

- **Tolon M.**, Yazgan U. and Ural D.; “An International Comparison of the 2011 Van Earthquake in the Meaning of Disaster Management Principles”, Proceedings of the International Van Earthquake Symposium, Van, Turkey, October 23 – 27, 2013. (Accepted, to be published).
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- **Tolon M.**; “A Comparative Study on Computer Aided Liquefaction Analysis Methods”, *International Journal for Housing Science and Its Applications*, Volume 37, no 2, pp.121-135, April 2013.
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