

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**ENDURANCE TIME METHOD APPLICATION IN NON-LINEAR SEISMIC
ANALYSIS OF FIXED MARINE STRUCTURE**

M.Sc. THESIS

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Department of Civil Engineering

Structure Engineering Programme

JUNE 2012

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**SABİT DENİZ YAPILARINDA ENDURANCE TIME YONTEMİNİN
LINEER OLMAYAN DEPREM ANALİZ KABİLİYETİ**

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To my spouse and parents,

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TABLE OF CONTENTS

	<u>Page</u>
FORWARD	ix
TABLE OF CONTENTS	xi
ABBREVIATIONS	xiii
LIST OF TABLES	xv
LIST OF FIGURES	xvii
SUMMARY	xxii
ÖZET	xxiii
1. INTRODUCTION	
1.1 Purpose of Thesis	2
1.2 Background	3
1.3 Hypothesis	6
1.3.1 General data	6
1.3.2 Foundation	6
1.3.3 Members	7
1.3.4 Weight and loads	8
1.3.5 Seismic endurance time method	8
2. INTEGRATED DATA.....	9
2.1 Codes and Standards Structures	9
2.2 Computer Model	10
2.2.1 Jacket	10
2.2.2 Top side and platform description	13
2.2.3 Appurtenances	16
2.3 Foundation.....	17
2.4 Weight Data and Design Loads	22
2.4.1 Substructures mass.....	22
2.4.2 Topsides mass	26
2.4.3 Crane loads	30
2.4.4 Helideck weight	31
2.4.5 Bridge load.....	32
2.4.6 Boat landing load	32
2.4.7 Area live loads	32
2.4.8 Marine growth	33
2.4.9 Wave and current load	33
2.4.10 Hydrostatic pressure.....	36
2.5 Member Section Properties	36
3. FLOW FORECASTING	39
3.1 Performance-Based Design Method.....	39
3.1.1 Performance levels.....	39
3.1.2 Performance-based design concept.....	46
3.2 Endurance Time Method	48
3.2.1 Endurance time concept.....	48
3.2.2 Accelerograms properties	53

3.2.3	Producing intensifying accelerogram functions.....	54
3.2.4	Accelerogram selection.....	62
3.3	Structures Analysis.....	64
3.3.1	Static pushover analysis.....	64
3.3.2	Time history analysis.....	64
3.4	Structures Performance Evaluation Based on FEMA356	64
3.5	Performance-Curve.....	67
3.6	Results of the Jacket Analysis	75
3.6.1	ET non-linear time history analysis output data	75
3.6.2	SPD25 jacket non-linear time history analysis output data under Seven modified accelerations	80
3.6.3	Result comparison.....	91
4.	CONCLUSIONS AND RECOMMENDATIONS	93
	REFERENCES.....	97
	APPENDICES	99
	APPENDIX A	100
	APPENDIX B	107
	APPENDIX C	113
	CURRICULUM VITAE.....	121

ABBREVIATIONS

A_i	: i^{th} Amplitude
API	: American Petroleum Institute
a_g	: ET accelero gram being sought
CD	: Chart Datum
CF	: Correction Factors
Cm	: Bending Moment Amplification Factor
COG	: Center Of Gravity
CP	: Collapse Prevention
DAF	: Dynamic Amplification Factor
DL	: Damage Level
E	: Modulus of elasticity
EL	: Elevation
ET	: Endurance Time
Fy	: Yield Stress for Steel Material
HAT	: Highest Astronomical Tide
I	: Moment of inertia
IO	: Immediate Occupancy
K	: Effective Length Factor
LAT	: Lowest Astronomical Tide
LS	: Life Safety
M	: Magnitude of earthquake in Richter
MDOF	: Multi Degree Of Freedom
N	: Return period of earthquake (years)
OP	: Operational
PGA	: Peak Ground Acceleration
PL	: Performance Level
R	: Structure response modification factor
SDOF	: Single Degree Of Freedom
$S_a(w)$	: Acceleration response spectrum
$S_a^T(w)$	: Target spectrum
$S_{ac}(T, t)$	: Codified design acceleration spectrum
$S_{aT}(T, t)$	: Target acceleration response
$S_{uT}(T, t)$	: Target displacement response
t	: Time
T_s	: Natural Period of the Structure
T_w	: Wave period
WHP	: Well Head Platform
w_i	: i^{th} angle frequency
α	: Optimization weight parameter
θ_y	: Yield rotation
ϕ_i	: Phase angle of i^{th} sinuous component

LIST OF TABLES

	<u>Page</u>
Table 1.1: Member effective length factor (K) and moment amplification reduction factor (C _m)	7
Table 2.1 : Piles and legs diaphragm constrained labels in SAP software	11
Table 2.2 : Conductor and casing dimensions	13
Table 2.3 : Summary of soil description at SPD25 location.....	18
Table 2.4 : Jacket appurtenance weight	23
Table 2.5 : Topside generated mass breakdown	27
Table 2.6 : Topside basic load cases weight breakdown	28
Table 2.7 : Crane loads	30
Table 2.8 : Area live loads	32
Table 2.9 : Marine growth profile.....	33
Table 2.10 :1-year directional maximum and significant storm tide and wave.....	34
Table 2.11 : 10-year directional maximum and significant storm tide and wave.....	34
Table 2.12 : 100-year directional maximum and significant storm tide and wave...	35
Table 2.13 : 1-year storm tide and design water depth.....	35
Table 2.14 : 100-year storm tide and design water depth.....	35
Table 2.15 : Member section properties	36
Table 2.16 : Design allowable stresses	37
Table 2.17 : D/T Ratio	37
Table 3.1 : Selected performance objective for a structure	42
Table 3.2 : Story drifts ratio range equal to performance levels for different Frames [6]	44
Table 3.3 : Plastic rotation range equal to performance levels for beam & Column [6].....	45
Table 3.4 : Description of the ground motion set used in this study	62
Table 3.5 : <i>a</i> and <i>b</i> values for world countries	69
Table 3.6 : Endurance time related to each PL	70
Table 3.7 : Story drift ratio of each PL	71
Table 3.8 : Beams acceptance criteria for nonlinear procedure in FEMA	73
Table 3.9 : Properties of used records and the CF of seven accelerations.....	80
Table 3.10 : Max. drift ratio table for seven time history analyses	86
Table 3.11 : Period and mass participation factor summary.....	89
Table A.1 : Jacket members in row 1	100
Table A.2 : Jacket members in row 2	100
Table A.3 : Jacket members in row A	101
Table A.4 : Jacket members in row B.....	101
Table A.5 : Jacket plan at EL 5.75m.....	102
Table A.6 : Jacket plan at EL -13m.....	103
Table A.7 : Jacket plan at EL -35m.....	104
Table A.8 : Jacket plan at EL -58.50m.....	105

Table A.9 : Jacket launch truss row A & B	106
Table C.1 : Equivalent Cross Section for Conductor.....	113
Table C.2 : Equivalent Cross Section for Pile & Leg.....	114

LIST OF FIGURES

	<u>Page</u>
Figure 2.1 : Diaphragm constrained nodes by SAP software	10
Figure 2.2 : Conductor	12
Figure 2.3 : Platform orientation	13
Figure 2.4 : SPD25 wellhead platform topside	14
Figure 2.5 : SPD25 wellhead platform topside, jacket, and substructures	15
Figure 2.6 : SPD25 modeled by SAP software	16
Figure 2.7 : 60 inch pile ultimate capacity at SPD25 location	19
Figure 2.8 : 48 inch pile ultimate capacity at SPD25 location	20
Figure 2.9 : Mud mat bearing capacity at SPD25 location	21
Figure 2.10 : SPD25 jacket operating COG plan view	24
Figure 2.11 : SPD25 jacket operating COG elevation	25
Figure 2.12 : Dummy rigid members	26
Figure 2.13 : SPD25 topside operating COG plan view	29
Figure 2.14 : SPD25 topside operating COG elevation view	29
Figure 2.15 : Crane DLB Abouzar 1200	30
Figure 2.16 : BELL helicopter	31
Figure 3.1 : Structural performance levels	40
Figure 3.2 : Structural performance levels & ranges	41
Figure 3.3 : Substance plastic behavior under lateral force [6]	46
Figure 3.4 : Performance-based design flowchart	47
Figure 3.5 : Hypothetical shaking table experiment	49
Figure 3.6 : Drift ratio of experienced frames under intensifying shaking	49
Figure 3.7 : Maximum drift ratio diagram for frame 2	50
Figure 3.8 : Structure response diagram under acceleration function	51
Figure 3.9 : Suggested intensifying accelerograms	53
Figure 3.10 : A typical Random Acceleration with PGA=1.	55
Figure 3.11 : f(t) Spectrum function	55
Figure 3.12 : Filtered frequency content	57
Figure 3.13 : Modified frequency content	58
Figure 3.14 : Response of modified accelerogram compared to codified values	59
Figure 3.15 : API & INBC code response	59
Figure 3.16 : Final intensifying accelerogram	60
Figure 3.17 : ETA20f02 acceleration function	60
Figure 3.18 : (a): ETA20f01 acceleration function, (b): ETA20f01 velocity and (c): ETA20f01 displacement function [6]	63
Figure 3.19 : Total acceleration response spectra of 7 scaled accelerograms and their Aaverage spectrum [6]	63
Figure 3.20 : Target curve procedure	67
Figure 3.21 : Endurance time evaluation diagram using return period	68
Figure 3.22 : Primary performance curve of drift	73

Figure 3.23 :	Modified performance curve	72
Figure 3.24 :	Maximum drift ratio performance curve	73
Figure 3.25 :	Performance curves for three plastic rotation modes	74
Figure 3.26 :	Average performance curve for plastic rotation.....	74
Figure 3.27 :	Maximum roof displacement in the X-direction obtained by TA20f	75
Figure 3.28 :	Maximum displacement in the X-direction obtained by ETA20f75	
Figure 3.29 :	Maximum story drift ratio in the X-direction obtained by ETA20f	76
Figure 3.30 :	Maximum story drift ratio in the X-direction obtained by ETA20f	76
Figure 3.31 :	Maximum story drift ratio compared with target curve	77
Figure 3.32 :	Maximum roof displacement in the Y-direction obtained by ETA20f.....	77
Figure 3.33 :	Maximum displacement in the Y-direction obtained by ETA20f78	
Figure 3.34 :	Maximum story drift ratio in the Y-direction obtained by ETA20f.....	78
Figure 3.35 :	Maximum story drift ratio in the Y-direction obtained by ETA20f	79
Figure 3.36 :	Maximum story drift ratio compared with target curve	79
Figure 3.37 :	Maximum story drift ratio obtained by LADSP record	81
Figure 3.38 :	Maximum story drift ratio obtained by LPAND record.....	81
Figure 3.39 :	Maximum story drift ratio obtained by LPGIL record.....	82
Figure 3.40 :	Maximum story drift ratio obtained by LPLOB record	82
Figure 3.41 :	Maximum story drift ratio obtained by LPSTG record	83
Figure 3.42 :	Maximum story drift ratio obtained by MHGO record	83
Figure 3.43 :	Maximum story drift ratio obtained by NRORR record	84
Figure 3.44 :	Average maximum story drift ratio of seven modified records ..	84
Figure 3.45 :	Average maximum displacement in the X-direction obtained by seven modified records.....	85
Figure 3.46 :	Average and 7 records Max. displacement in the X-direction.....	85
Figure 3.47 :	Maximum drift ratio in the X-direction obtained by pushover	87
Figure 3.48 :	Maximum displacement in the X-direction obtained by pushover...	87
Figure 3.49 :	First mode of modal analysis	88
Figure 3.50 :	Compared maximum story displacement in X-direction	91
Figure 3.51 :	Compared maximum story displacement in Y-direction	91
Figure 3.52 :	Compared maximum drift ratio in X-direction	92
Figure 3.53 :	Compared maximum drift ratio in Y-direction	92
Figure B.1 :	Jacket elevation plan raw 1	107
Figure B.2 :	Jacket elevation plan row 2.....	108
Figure B.3 :	Jacket elevation plan row A.....	108
Figure B.4 :	Jacket elevation plan row B	109
Figure B.5 :	Framing plan at EL 5.75m	109
Figure B.6 :	Framing plan at EL -13m.....	110
Figure B.7 :	Framing plan at EL -35m.....	110
Figure B.8 :	Framing plan at EL -58.50m.....	111
Figure B.9 :	Jacket leg plan.....	111
Figure B.10 :	Jacket pile plan	112
Figure B.11 :	Conductor plan	112
Figure C.1 :	Mode 1	115
Figure C.2 :	Mode 2.....	116

Figure C.1 : Mode 1	115
Figure C.2 : Mode 2	116
Figure C.3 : Mode 3	117
Figure C.4 : Mode 4	118

ENDURANCE TIME METHOD APPLICATION IN NON-LINEAR SEISMIC ANALYSIS OF FIXED MARINE STRUCTURE

SUMMARY

"Performance-Based Design" is a new method for designing of the structures so that to predict and control their performance in the earthquakes with attended magnitudes. This method is rapidly developing in modern earthquake and civil engineering and has demonstrated its potential ability in the structural design. In this method, predicted performance level of a given structure after construction, is highly considered as an important design parameter. For this purpose, a performance objective (level of performance) must be defined. A performance objective is defined as an acceptable damage level for the structure facing an attended magnitude of earthquake. On the other hand, the dynamic analysis methods such as "Time History Analysis" in linear and nonlinear situations provide more accurate results. However, some issues such as record selection techniques, output diversities and complexity of the results, are convincing reasons to choose the traditional simple methods. The new "Endurance Time method" is a dynamic analysis-based method that relates the results to the time, which results in a reduction in complexities due to the type of analyses. In addition, the Endurance Time method resolves some confusion that might exist in other methods such as the "Performance-Based Design" method.

In this project, we first study the FEMA's regulation regarding to the Endurance time method, which is one of the most valid regulations in the analysis and design of structure. Then by defining a concept called "performance curve", we study the application of Endurance Time in defining the performance level of a jacket type structures. Finally, the results obtained from Endurance Time analyses of the jacket SPD25 are compared with the FEMA performance curve and the results that are obtained from seven time history analyses and pushover analyses.

Keywords: Performance-Based Design; Endurance Time Method; Performance Levels; Time History Analysis; Fixed Marine Structure; Jacket

SABİT DENİZ YAPILARINDA ENDURANCE TIME YONTEMİNİN LİNEER OLMAYAN DEPREM ANALİZ KABİLİYETİ

ÖZET

“Performansa Dayalı Tasarım” yapıların kontrolü için ve bu yapıların beklenen depremlerde performansını incelemek için gelişmiş bir yöntemdir. Bu yöntem çok hızlı bir şekilde modern deprem ve yapı mühendisliklerinde kullanılmaktadır ve yapı tasarımında önemli bir rol oynamaktadır. Bu yöntemde, mevcut yapının inşaat sonrası tahmin edilen performans düzeyi, önemli bir tasarım parametresi olarak kabul edilir. Bu amaçla belli bir performans düzeyi tanımlanmaktadır. Bu performans düzeyi yapının depremde beklenen hasar düzeyini göstermektedir. Diğer açıdan, dinamik analiz yöntemleri örneğin lineer “Time history” analizi lineer daha doğru sonuçlar vermektedir. Buna rağmen, bazı durumlar; kayıt seçme teknikleri, çıktıların çeşitlilikleri ve sonuçların karmaşıklıkları basit yöntemlerin kullanılmasını gerekli kılar. Geliştirilen bu yeni ET yöntemi, direkt zamana bağlı analiz sonuçları verdiğinden, kullanılan analiz yöntemlerinden kaynaklanan komplikasyonları azaltır. Buna ek olarak, diğer bazı yöntemlerdeki ki başka problemlerde (örn. Performansa dayalı tasarım yöntemi) azaltmaktadır.

Bu çalışmada önce FEMA yönetmeliği, tasarım açısından incelenmektedir. Sonra “performans eğrisi” ve “tahrip düzeyi indeksi” kavramları acılanmaktadır. Son olarak, SPD 25 petrol platform yapısının performans eğrileri belirlenmiş ve sonuçlar ET analizleriyle karşılaştırılmıştır.

Yapıların deprem sırasında sismik performansı hassas konuları biridir ke yapıların güvenliğini ve ekonomik koşullarıyla ilgili. Günümlerde, makul bir düzeyde güven ile ilgili economic kararlar için, genellikle sahipleri yapılarını performansını bir deprem sırasında bilmek istiyorlar. Bu faiz delivering bir deprem sırasında öngörülebilir bir performans kapasitesine yapıları tasarlama yöntemlerini geliştirmek için mühendisler açmıştır.

Performansa dayalı Deprem Mühendisliği aslında çeşitli yordamlar, sayede yapısı için kabul edilebilir bir sismik performansı sağlanmaktadır oluşur. Yordamı, tehlike düzeyi site tanımlama, geliştirme kavramsal, ön ve nihai yapısal tasarım, inşaat ve bakım onun ömrü boyunca bir binanın içerir [1].

Deprem engineering bilginin geliştirilmesi ve estimating için daha güvenilir yöntemler geliştirilmesi ile büyüklükleri farklı sitelerdeki yapıların sismik analiz yöntemleri olan deprem konusu clude daha karmaşık parametreler evaluation etkileri ve yapıların sismik tehlike analizi geliştirdi.

Öte yandan, "Time history" ve "Statik analiz" gibi dinamik analiz yöntemleri doğrusal ve doğrusal olmayan durumlarda kabul bir değeri vardır. Ancak kayıt seçim teknikleri, çıkış farklılıkların ve sonuçları karmaşıklığı, gibi bazı sorunları geleneksel basit yöntemlerle seçmek için convincible neden olabilir. Yeni "Endurance Time method" özel tasarlanmış intensifying accelerograms, onlar empoze dinamik uyarılmalar sırasında karşı koyabilir zaman aralığına dayalı olan

süre ile orantılı olarak artış ve dayanıklılık süresi yapıları ölçülür yanıt spectra bir dizi yapıları tabi tutulmaktadır.

Bu tez, et yöntemi uygulayarak ve dinamik analizi, tasarım, performansa dayalı bir ceket düşüncesi bir ceket sismik davranışı araştırdı. et yordam temelleri kısaca gözden geçirilir. Doğrusal olmayan analiz, ceket, altında et ivme işlevler, dikkate alarak dayanıklılık süresi analizleri sonuçları verilmiştir ve statik oyuncağı analiz karşılaştırılır. Bir ceket sismik davranışı mantıklı yaklaşım et yöntemiyle tarafından öngörülen gösterilir.

NEHRP Provisions için yeni binalar ve diğer yapılar (bssc, 1997) deprem yönetmeliği göre doğrusal olmayan [3] ve FEMA 273 nehrp yönergeleri binaların deprem rehabilitasyon (bssc, 1997) [4], üç performans düzeyleri (PLs) kabul edilir. Bunlar, Immediat Occupancy (IO), Life Safety (LS) ve Collapse Prevention (CP) denir.

İki ara yapısal performans aralığı ek olarak bahsedilen üç düzeyi vardır: a) hasar Control b) sınırlı güvenlik. Hasar denetim düzeyini hasar durum LS yapısal PL ve IO yapısal PL [12] arasında sürekli aralığı olarak tanımlanmış. Bu aralıkta yapısı elastik genellikle modundadır. Sınırlı güvenlik düzeyi olarak hasar durum LS yapısal PL ve CP yapısal pl [12] arasında sürekli aralığı tanımlanması.

İlk hasar durumunda Immediat Occupancy (IO) yanal direnci oluştu veya sadece küçük yapısal hasar yerçekimi bina görüntü ve hiçbir önemli azalma vardır. Yapısında önemli hasar meydana geldi, ancak Collapse Prevention (LS) düzeyinde yapı elemanları çöktü önlemek için yeterli yeteneği vardır. Collapse Prevention (CP) düzeyi kritik hasar oluştu ve yapısını çöktü [4] yaşandığı eşğinde olduğunu post-earthquake hasar durumu olarak tanımlanır.

Performans düzeyleri genellikle bir yapıya zarar spektrumunda ayrı bazı şeritler halinde gösterilir ve yapısal ve mekansal bileşenlerine hasar miktarı yapılarına göre hasar durumunu bölebilirsiniz. Ayrıca, maliyet gibi diğer bazı kavramları kez onarım ve yaralanma performans düzeyleri, çok ilgili.

Başka bir pratik kavramı "performans için sismik tehlike belirli bir düzeyde bir yapısal performans düzeyi (örneğin çöktü önleme, Life Safety Life Safety veya Immediat Occupancy) tayini oluşun", hedefidir. et yönteminde, böylece hesaplamalı talebi önemli ölçüde azaltarak bir tek zaman-geçmiş analizi, farklı excitation düzeylerde yapısal yanıt alınır. et yöntemini kullanarak ve performansa dayalı tasarım kavramları ile ilgili olarak, bir yapının sismik tehlike çeşitli düzeylerde performansı tek bir zaman-geçmiş analizinde, diğer bir deyişle, bir yapısı tek bir time history analiz performans hedeflerine tatmin olmadığını tahmin edebilirsiniz öngörülen.

Bu projede, Endurance Time yöntemi uygulamaya performansa dayalı tasarım alanının genişletilmesi için yeni bir yöntemi tanıtdı. Aslında, et yöntemi karakteristik en iyi performansa dayalı tasarım temel kavramlarını birleştirmek için onun kavramları genişleterek yararlı olabilir. Endurance Time ilgili performans düzeylerini accelerogram işlevler için tanımlanır. Bahsedilen modellenen ceket, yukarıdaki yöntemleri uygulamak için kullanılır. Nihayet dayanıklılık süresi yöntem sonuçları zaman Geçmiş ve oyuncağı analizi ile karşılaştırılır.

Deprem etkileri tabi olduğunu zaman geçirerek değişir, gerçek öteleme yapısı ortaya çıkar. Bu sismik etkileri nedeniyle, yapısal üyeleri güçleri ve gerilmeler yapılacaktır. Stresleri elastik aralığı düşük bir sismik hareket ve hiçbir zarar yapısı vardır. Yine de, bu stresleri daha elastik aralığı olsaydı, hasar çatlama, smashing, bükülüyor ya

da verim bir türlü bazı üyeleri oluşacak. Bu nedenle stresleri bazı üyeleri başarısız olmasına neden ve sismik hareket artırarak yapısının neden toplam çöküşü.

Endurance Time analizi, yapısal performans sağlayarak açıklanacaktır et yöntemi büyük hesaplamalı çaba structures pratik performans tasarımı için gerekli önemli ölçüde azaltabilirsiniz. Stres testleri, hasta bir değişken yamaç. Test bir düşük eğim ve düşük hızlı koşulu ile başlar. Gibi kan basıncı, kalp atım hızı, vb. izlendiğini sınama sırasında eğimi ve hızı yavaş yavaş, hasta, fiziksel ve biyolojik koşulu artmış bulunmaktadır. Test, sıkıntı veya anormal koşulları belirtileri gözlenen kadar devam eder. Daha sonra kalp rahatsızlığı tolere hızı ve yamaç düzeyi üzerinde göre değerlendirilecektir.

Endurance Time yönteminde, neredeyse aynı kavram uygulanır. Arkasında et yöntem kabaca, yapısı üzerinde bir rampa gibi accelerogram koymak ve ne kadar gidebilir görmek olur. Yapı dinamik talep yoğunlaştırılması ile standart bir accelerogram tabi. Until they reach tanımlanmış bir maksimum değer belirtilen performans Endeksleri izlenmektedir. Performans yapısı zaman zaman hasar sınırı dizini aşıldı göre değerlendirilecektir.

Endurance Time yöntemi (et) kavramı, varsayımsal bir experiment dikkate alarak en iyi açıklanabilir. Bilinmeyen sismik direnci özelliklere sahip üç model binalar investigated referans ile performanslarını karşı yüksek yoğunluklu depremler çöküşü olarak varsayın. Şimdi bu modeller ve titreşim masaya koymak için düzeltilen düşünün. Deneme binalar rasgele titreşim yavaş yavaş increasing şiddette subjecting tarafından başlatılır. Böylece tüm üç bina titreşimle ancak stabil başlangıçta sallayarak, genlik oldukça düşük. Titreşim genliğini arttıkça, bir bina çöker zaman bir noktaya ulaşılır. Bir modeli varsayılır. Titreşim genlik zaman içinde daha fazla arttıkça ikinci yapısı başarısız olur.

Bu karakteristik et yönteminin en iyi performans dayalı tasarım temel kavramlarını birleştirmek için onun kavramları genişleterek kullanılabilir ve sonra ceket, bu tezin ana amacı olan varolan 3B modeli uygulanır. Ayrıntı, bu önerilen metodoloji açıklandığı gibi "Performance Curves" tanıtıldı. Gutenberg-Richter denklemler gibi bazı denklemler kullanan her pl için karşılık gelen zaman eşdeğer dayanıklılık elde. Yeni tanıtılan "Referans performans eğri" veya "Başvuru excitation düzeyini sürekli bir fonksiyonu olarak gerekli performansı gösteren eğri" çizilmiş [5].

Gerçek performans sonra et Analiz sonuçlarına göre bu hedef performans karşı çizilir. Yapının genel performansını gerçek performans hedefi karşılaştırarak beklenen ve tasarım göre gözlenen performansı artırılabilir. Sonunda çelik anı çerçeveleri gibi davranır, bir ceket, performansı referans eğrisi kullanılarak değerlendirilir ve sonuçlar zaman Geçmiş ve statik oyuncağı analiz sonuçları ile karşılaştırıldığında. Gibi iyi bir es-timate hedef zaman Time history analizi, yapısal performans sağlayarak açıklanacaktır et yöntemi büyük hesaplamalı çaba structures pratik performans tasarımı için gerekli önemli ölçüde azaltabilirsiniz.

1. INTRODUCTION

Seismic performance of structures during strong earthquakes is one of the most sensitive considerations, regarding their safety and economic requirements, to be set as their design criteria. Nowadays, usually the owners like to know the performance of their structures during an earthquake in order to make relevant economic decisions with a reasonable level of confidence. This interest has led engineers to develop methods for designing structures that are capable of delivering a predictable performance during an earthquake. Performance-based earthquake engineering essentially consists of various procedures, whereby the structure is ensured for an acceptable seismic performance. The procedure involves identification of the hazard level for the site, development of conceptual, preliminary, and final structural designs, construction, and the maintenance of a building during its lifetime [1].

With the improvement of knowledge in earthquake engineering and the development of more reliable methods for estimating earthquake magnitudes at different sites, methods for the seismic analysis of structures are being developed that include the effects of more complicated parameters in the evaluation and seismic hazard analysis of structures. [2]

On the other hand, the dynamic analysis methods such as "Time History Analysis" and "Static Analysis" in linear and nonlinear situations have a considered value. However, some problems such as record selection techniques, output diversities and results complexity, are convincing reasons to choose the traditional simple methods. In the new "Endurance Time method" structures are subjected to a set of specially designed intensifying accelerograms, the response spectra of which increase proportionally with time, and the Endurance Time of the structures are measured based on the time interval during which they can resist the imposed dynamic excitations.

In this thesis, applying the ET method and considering the dynamic analysis of a jacket using Performance-based design, Seismic Behavior of a jacket are investigated.

The basics of the ET procedure are briefly reviewed. In non-linear analysis, the results of the Endurance Time analyses of Jacket, taking into account under ET acceleration functions, have been presented and are compared to the static pushover analysis. It is shown that the seismic behavior of a jacket can be predicted by the ET method with reasonable approximation.

1.1 Purpose of Thesis

As per FEMA-302 NEHRP Recommended Provisions for the Seismic Regulations for New Buildings and Other Structures (BSSC, 1997) [3] and FEMA 273 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (BSSC, 1997) [4], three Performance Levels (PLs) are considered. These are termed Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). In the first damage state Immediate Occupancy (IO), only minor structural damages are visible and no substantial reduction in building gravity or lateral resistance has occurred. In the Life Safety (LS) level, although significant damage to the structure has occurred, structural elements have enough capability to prevent collapse. The Collapse Prevention (CP) level is defined as the post-earthquake damage state in which critical damages are occurred and the structure is on the verge of experiencing collapse [4]. Another practical notion is "Performance Objective", which consists of the specification of a structural performance level (e.g. collapse prevention, life safety, or immediate occupancy) for a given level of seismic hazard.

In the ET method, the structural responses at different excitation levels are obtained in a single time-history analysis, thus significantly reducing the computational demand. So using the ET method and regarding the concepts of performance-based design, the performance of a structure at various levels of seismic hazard can be predicted in a single time-history analysis, in other words, one can estimate if the structure satisfies its performance objectives by only one time-history analysis.

This characteristic of the ET method can be best utilized by extending its concepts to incorporate the basic concepts from performance-based design and then is applied to the existing 3D model of jacket, which is the main purpose of this thesis.

As will be explained in detail, in this proposed methodology, the "Performance Curves" is introduced. Utilizing some equations, such as Gutenberg-Richter equations, the equivalent endurance times corresponding to each PL are obtained. A newly introduced "Reference Performance Curve" or "Reference Curve" representing the required performance as a continuous function of the excitation level is drawn. [5]

The actual performance is then plotted against this target performance based on the results of ET analysis. The overall performance of the structure can be anticipated by comparing the target to the actual performance and the design can be improved based on the observed performance.

Finally, the performance of a jacket, which behaves such a steel moment frames, is evaluated using the reference curve and the results are compared with Time history and static pushover analysis results. As will be explained, by providing a good estimate of structural performance at target time in time-history analysis, the ET method can considerably reduce the huge computational effort required for the practical performance design of structures.

1.2 Background

In this paper, the nonlinear time-history based dynamic pushover Endurance Time (ET) method is applied to a set of 2D steel moment-resisting frames with a different number of stories and spans. Before these frames are subjected to gradually intensifying dynamic acceleration functions, which are compatible with Iranian National Building Code (INBC 2800) design spectrum and support nonlinear ranges, the accelerations are calibrated and then performance levels (PLs) are clearly defined. After running the response history analysis, the ET curves corresponding to response parameters, such as reference curve and performance curve are created and Peak Ground Acceleration (PGA) is obtained based on the curves. Finally, Performance Curve (PL) and Damage Level (DL) are incorporated and one combined damage indicator has been defined for the purpose of the paper that reduces the creation of a target curve for each element. [5]

The seismic static analysis is a simple proposed method in the standards for designing the structures against the earthquake and the ET method is a new analysis, which has no some limitations of static analysis. In this article, steel frames from one to 12 stories are studied by static analysis and gradually intensifying dynamic acceleration functions which are calibrated. Then, the results of the acceleration functions are compared with results of static method analysis. Finally, the compared results shows the capability and accuracy of time history ET method, and the static analysis in stresses and drifts are depicted in various points of structures. In addition, time history method has more versatility of spectrum method than static analysis and predicts more optional dimensions for structural elements. [6]

In this paper, the structure is subjected to the ET accelerograms that imposing dynamic demand on the structure with time. The procedure of choosing the appropriate dynamic input and calibration of them are explained. Then, these accelerograms are applied to the SDOF and MDOF systems and finally the drifts are monitored based on the ET method. [15]

In this paper, application of the Endurance Time (ET) method in seismic analysis of concrete gravity dams has been investigated. The ET method is based on the time history analysis of the structures, subjected to the specially designed intensifying acceleration functions. It is expected that by developing its application in analysis of the concrete dams, the useful information on the seismic behavior of such dams at various excitation intensities can be obtained. Results from linear analysis of Folsom and Koyna dams under real earthquakes and ET acceleration functions have been compared. It is shown that the ET method can predict the response of concrete gravity dams to individual earthquakes with reasonable accuracy in linear analysis. In order to evaluate concrete gravity dams by linear analysis, the target time has been assumed to correspond to the OBE level of earthquake. Endurance criteria can be set as the time when the desired damage indexes, e.g. maximum principal stress, reaches its allowable value. The design can be considered satisfactory if endurance time is more than target time and vice versa. In addition, potential application of the ET method in non-linear analysis of concrete gravity dams has been investigated. [2]

In this paper, the structures with different bays and stories are subjected to the ET intensifying ground acceleration functions and their performance is judged based on their response at various excitation levels. Response spectra of seven ground motions on the stiff soil are used to produce intensifying acceleration functions. The drift ratios and plastic hinge rotations are compared well with those from ground motions in steel frames. The locations of plastic hinges are also predicted quite satisfactorily by ET analysis. Finally, the validity and the applicability of the ET method in the nonlinear seismic analysis of steel frames are assessed and are compared with the results of the pushover and time-history analysis. [21]

In this study, some fundamental concepts of ET method are explained and the potentials and limitations of this procedure in nonlinear seismic analysis of SDOF structures are investigated. Numerical optimization procedures for generating ET acceleration functions that are compatible with ground motions are explained. Results of ET analysis for inelastic SDOF systems are compared with ground motions analysis results for different strength ratios, ductility, and damping ratios. The accuracy of ET method in predicting the response of SDOF systems with stiffness degradation and strength deterioration is also investigated. Application of ET method in performance based earthquake engineering is described by an example of a single degree of freedom system. The results show that the approximations of ET method are in a good agreement with the exact response history results of the similar ground motions for different nonlinear systems. It is shown that ET acceleration functions optimized in the linear range considering long periods can be used in the nonlinear analysis with reasonable accuracy. [22]

In this study, one of the potential applications of the ET method is in the three-dimensional analysis of buildings considering multidirectional excitation. In this paper, a procedure for multi-component analysis by the ET method has been developed. Several steel moment frames with heights from 9.6 to 48m were designed according to the Iranian National Building Code. By applying the proposed algorithm for three-dimensional analysis of structures by the ET method, these frames were analyzed, and results have been compared with static, response spectrum and response history methods considering typical code recommendations. Results show that for all studied models, even when irregularity and height of structure increase, the ET method estimates the seismic response of steel frames with reasonable accuracy compared with other investigated approaches. [23]

1.3 Hypothesis

In this part, the brief and significant definitions of a model and method are explicated and the details will be cleared in their own seasons. This thesis expresses the applied nonlinear dynamic ET method on the study case SPD25 Wellhead Platform (WHP) from the South Pars Field, located offshore, some 100 km Southwest of Asaluyeh, Persian Gulf, Iran.

1.3.1 General Data

The system of measurement shall be the System of International (SI), the datum for the wellhead platform is the center of the wellhead arrays, and all elevations are referenced to Lowest Astronomical Tide (LAT). All structural analysis shall be performed in SAP software version 14. The 3D SAP model of jacket will include structural members, the pile foundations, and schematic mass less sections of topside that all weight of topside is applied in the Center Of Gravity (COG). Non-structural members such as boat landing, risers, J-tubes, and conductors will be modeled for hydrodynamic loads but not affect the structural stiffness.

1.3.2 Foundation

The foundation model shall be based on the soil design parameters (p-y, t-z, and q-z) in the Fugro geotechnical report. [7]

The pile can be assumed fixed at a certain distance below the seabed as following and because the soil and pile interaction is not the subject of this thesis, it is modeled based on the pile and leg plan (Appendix B).

- 3.5 D-4.5 D for stiff clays
- 7 D-8.5 D for very soft silts
- 6 D for general calculation

1.3.3 Members

Structural members will be represented by element with equivalent axial and bending stiffness properties. The elements will connect structural joints from work-point to work-point. The model shall include wall thickness and diameter changes at nodes. Composite section with equivalent axial and bending stiffness properties will be modeled for the grouted piles inside the jacket legs. The conductors' section properties are changed to an equivalent section by equivalent-cross equations so that the moment of inertia and section module are equal or more. In fact, the gross weight of equivalent-cross section must be as same as primary section because the conductors are nonstructural elements. Effective length factor (K) and bending moment amplification factors (Cm) are as bellows: [API RP2A WSD 3.3.1.e] [9].

Table 1.1: Member Effective length Factor (K) and Moment Amplification Reduction Factor (Cm)

Member	K factor	Cmy/Cmz
Jacket legs	1.0	0.85
Jacket horizontal members not framing into jacket	0.8	0.85
Jacket horizontal diagonal members framing into legs	0.8	0.85
Jacket braces		
-diagonals	0.8	0.85
-X braces	0.9	

1.3.4 Weight and Loads

Based on the analyze method in this thesis (ET method) and API recommendations, the required loads shall be gravity, buoyancy, and hydrostatic pressure. [API RP2A WSD 2.3.6.c] [9]

So that, the supposing loads and weight are as following:

Jacket self-weight, jacket buoyancy, appurtenances weight, marine growth, topside operating weight, bridge load, crane operating load, helideck and boat landing which jacket self weight and buoyancy will be generated automatically by the SAP Program with a seawater density of 1.023 kg/m^3 .

1.3.5 Seismic Endurance Time Method

To start an ET time history analysis, a suitable damage measure and an appropriate ET accelerogram must be set. The calibrated accelerograms is series of three accelerograms optimized to match response spectrum of selected ground motions. These records have been made to match ground motions average response spectrum in the long period range and therefore produce reasonable estimates of response in nonlinear analysis. [Urf-1]

The design response spectrum is the API response spectrum for type C soil (Type II in INBC: Iranian National Building Code) with the peak ground acceleration which will be obtained in the next sections. A critical damping factor of 5% was used in the analysis.

The masses will be lumped at the primary joints such that the center of mass corresponds to the COG of the topside and jacket. An added mass coefficient of 1.07 be used for all tubular members to generate the added mass associated with all submerge members.

2. INTEGRATED DATA

In this section, the codes, and standards, jacket SAP 3D model, design data, design loads and criteria will be defined in details, whereas this thesis is study case the most data is based on Iranian Offshore Engineering and Construction Company (IOEC) documents.

2.1 Codes and Standards Structures

The structure shall be designed in accordance with editions and the related amendments of the following:

- API-RP2A WSD 2000 2A_WSD: Recommended Practice for Planning, Design and Constructing Fixed Offshore Platform Working Stress Design, 21st Edition, on Dec.
- AISC: Manual of Steel Construction Allowable Stress Design, Ninth Edition
- FEMA-356 & FEMA-440 NEHRP: Recommended Provision for Seismic Regulations for New Buildings and Other Structures.
- NIOC South Pars Field Development Structural Material Specification Document No SP17/18-P2-ST-SP-8028
- NIOC South Pars Field Development Project Phases 17 &18 Offshore Platform Document No SP1718-P3-ST-CS-8202 REV 2
- Fugro Survey (Middle East) Ltd. Engineering Report South Pars Gas Field Development Phase 17 & 18 Geotechnical Soil Borings South Pars Field offshore Iran Rev 3 April 2007
- N.D.P.Barltrop, A.J.Adams, Atkins Oil & Gas Engineering Limited, Epsom, UK,Dynamic ,1991 Third Edition. Dynamic of Fixed Marine Structures 1&2 Linacre House, Jordan Hill, Oxford OX2 8DP

2.2 Computer Model

A three-dimensional jacket and schematic mass less sections of topside SAP model developed for the in-place analysis was used for ET seismic analysis.

The components of the SAP geometry model are:

2.2.1 Jacket

2.2.1.1 Jacket Legs and Piles

The jacket leg spacing at the work point shall be 13.716m x 20.0m. The grouted pile and jacket leg are modeled as a composite section of the pile and jacket leg diameter and wall thickness. Effective wall thickness for grouted jacket leg and piles will be in accordance with API RP 2A WSD Supplemental 2 C5.3.4. The pile head joints are modeled at EL (-) 58.50 and EL (+) 7.24. A pile stub (1524 mm x 76.2 mm w.t) is modeled between the pile head joints and jacket at the mud line EL (-) 58.50 m. The pile and leg joints at each four elevations are modeled as diaphragm nodes, which are constrained at Z-axis. In other word, the pile and leg will act un bloc at X and Y directions.

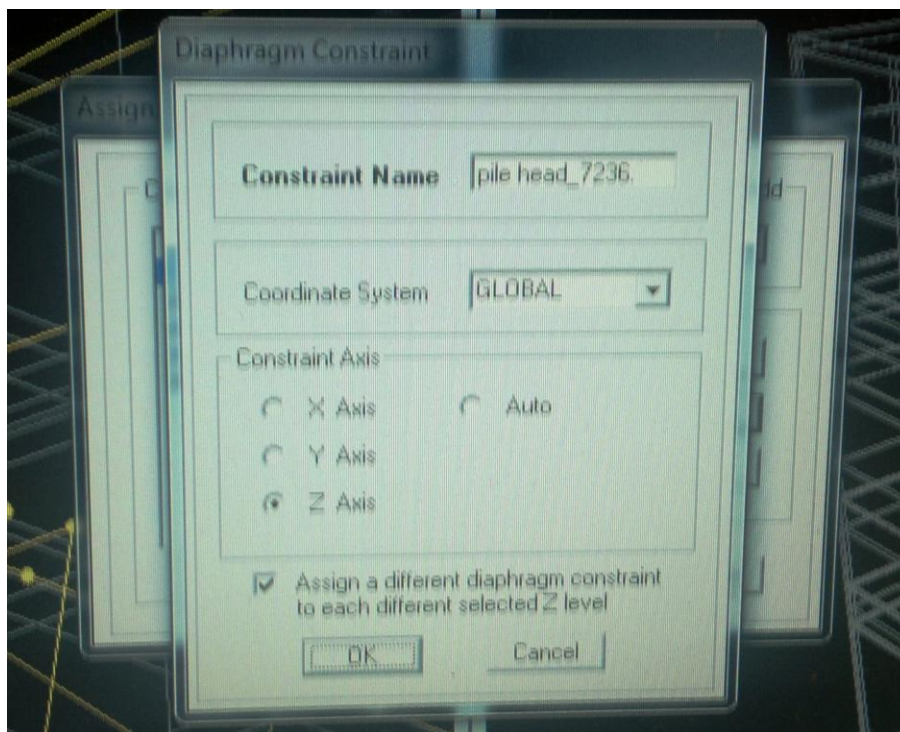


Figure 2.1 : Diaphragm Constrained nodes by SAP software

These piles and legs nodes label are as below:

Table 2.1 : Piles and Legs Diaphragm Constrained Labels in SAP Software

Row-Column	EL + 7.24	EL -13.00	EL -35.00	EL -58.50
A-1	383	486	483	363
A-2	386	411	484	379
B-1	381	155	482	350
B-2	388	404	481	356

Column buckling effective length factor (K) for legs shall be based on Table 1.1 recommendations. The legs section properties will be mentioned in design criteria part and the jacket elements are in Appendix A and the plan is in Appendix B.

2.2.1.2 Horizontal Framing

The horizontal frame elements in four different elevations are modeled based on the plan, and the buoyancy load is considered where air in horizontal elements is.

2.2.1.3 Elevation Bracing and Diagonals

Each side of a jacket has three bracing spans that in initial stages of brace deformation subjected to vessel impact, the response, which are affected by local denting and buckling, is controlled by bending effects under the load. The kinetic energy is absorbed by global platform displacement, dent formation, deformation of brace due to bending energy. The tubular joints shall provide sufficient and adjacent capacity to transfer the brace reaction forces. The allowable stress for design of joints and adjacent members shall be the material yield strength. [10]

Gap between braces has been defined using member offset. Member end offsets have also been defined at the leg nodes to reflect the real length and stiffness of braces to the jacket legs.

For impact on brace, the most critical diagonal brace shall be designed as a minimum with due consideration for different brace size.

These brace member are modeled based on the plan exactly. Bending moment amplification factors (C_m) are explicated in table 1.1

2.2.1.4 Conductors

Each wellhead platform shall have fifteen conductors. The conductors shall be in an array of three rows of five. Spacing conductors will be 2286 mm and 2594 mm transversely and 1830 mm longitudinally. The conductors are 660 mm diameter. The conductors are modeled as non-structural member with four supporting connection to the jacket at each framing level. The supporters have member end released such that only lateral forces can be transferred between the conductors and framing guide. The conductor and casings exist in plan [Appendix B] which conductors are modeled with an equivalent cross section with density of 24.62 t/m^3 for the casings and grouted weight. [Appendix C]

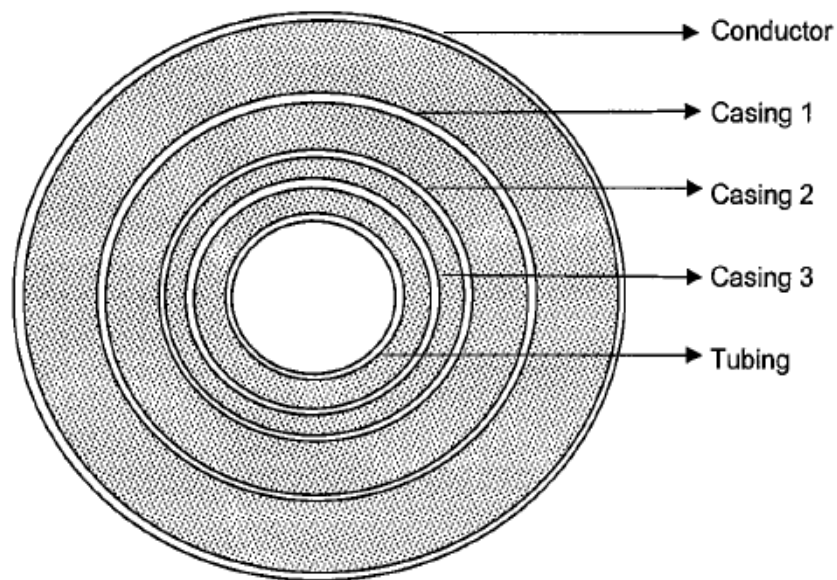


Figure 2.2 : Conductor

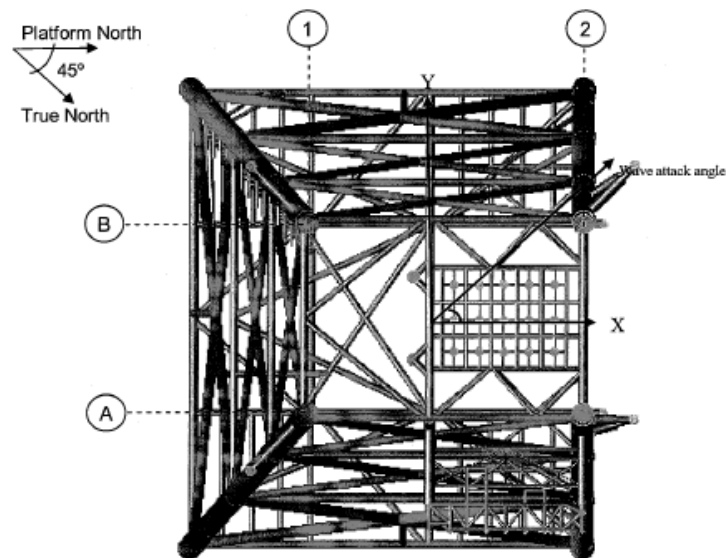
Table 2.2 : Conductor and Casing Dimensions

Item	Diameter (mm)	Wall thickness(mm)	Area (mm ²)	Unit weight (MT/m)
Conductor	660.4	25.40	50670.7	0.398
Casing 1	473.1	11.05	16039.9	0.126
Casing 2	339.7	9.65	10005.9	0.079
Casing 3	273.1	8.89	7379.1	0.058
Tubing	193.7	8.33	4851.0	0.038

2.2.2 Top side and Platform Description

The WHP substructure is a four-leg tubular steel template structure with battered jacket leg. The top side shall be configured to accommodate 15 well slots, equipments, a technical room, an accommodation room, and a crane. We just modeled the weights of the top side components in their center of gravity because of a top side data deficiency. So that, the results may differ with real one but whereas the concept of the thesis is comparison of two ET and static pushover analysis, it would not obstruct it. Platform north is 45 degrees west of True North. The computer model is located at LAT EL +0.00 at the center of the platform. The computer model axes systems are:

(+) X Platform North, (+) Y Platform West, (+) Z Vertically Up

**Figure 2.3 : Platform Orientation**

The top side dead operating weight, which includes 75% of the storage loads as per API RP2A, 2.3.6c, bridge dead weight, and live load are modeled in the top side module operating COG plan.

The topside operating weight in the analysis shall be 2820T, plus 151T for bridge, 24T for crane operating load, 13000 lb for helideck, 72T for boat landing, and 31T for live loads. This compared with the weight and loads section. [10]

The top side and jacket framing elevations are:

- Top deck with a helideck EL (+) 27.40m
- Upper deck EL (+) 22.90m
- Lower deck EL (+) 18.90m
- Cellar deck EL (+) 13.90m
- Drain deck EL (+) 11.100m
- Top of jacket EL (+) 7.24m
- Level 1 EL (+) 5.750m
- Level 2 EL (-) 13.00m
- Level 3 EL (-) 35.00m
- Level 4 EL (-) 58.50m

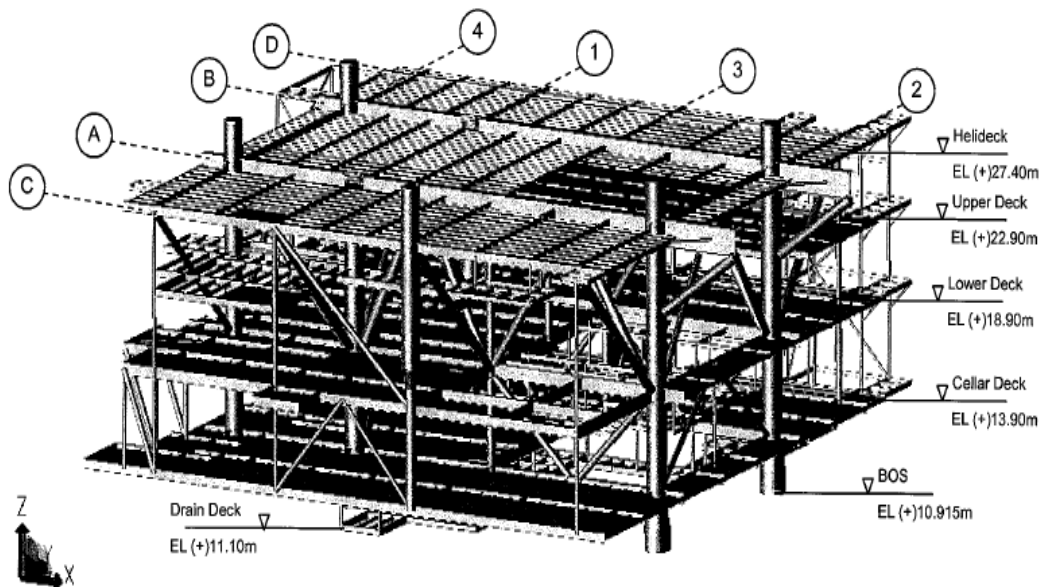


Figure 2.4 : SPD25 Wellhead Platform Topside



Figure 2.5 : SPD25 Wellhead Platform Topside, Jacket, and Substructures

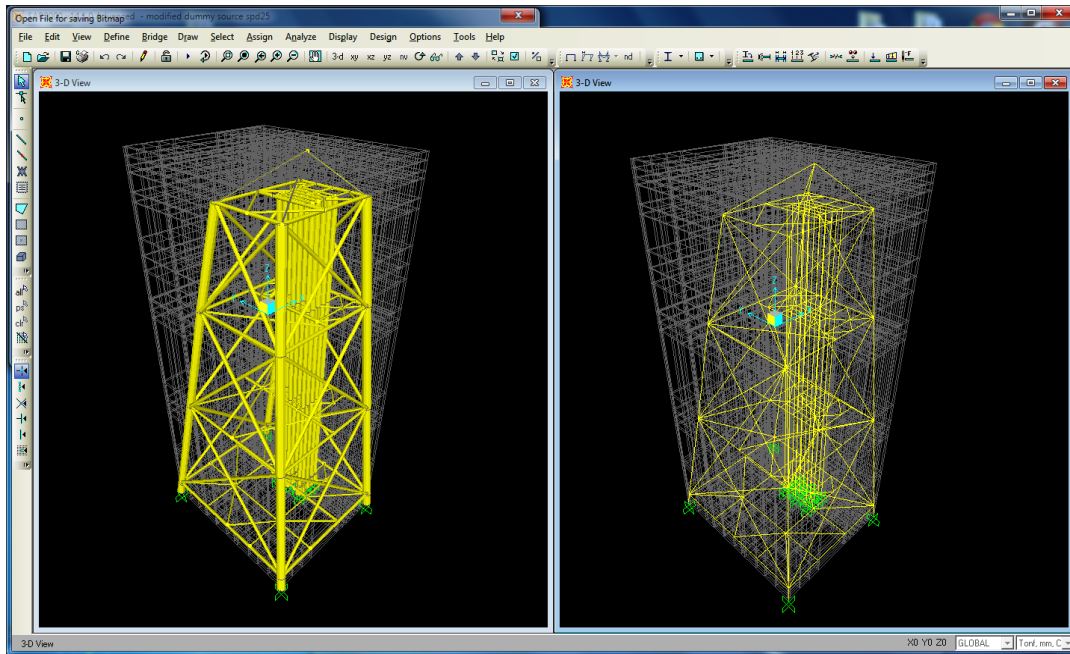


Figure 2.6 : SPD25 Modeled by SAP Software

2.2.3 Appurtenances

Appurtenances have been modeled explicitly. All non-modeled appurtenances such as jacket walkway grating, handrail, launch cradle, anodes, and conductor guides are counted as weights in dead load case. The added mass coefficient of 1.07 is counted for sacrificing anodes weight. Elastic and shear modulus of the boat landing, caissons, risers, and J-tubes has a minimal value such that its stiffness is not accounted for the jacket stiffness. All modeled appurtenance masses were converted from load to lump masses at the respective joints in the SAP model. The appurtenances explicitly modeled are explained in the following.

2.2.3.1 Boat landing

One boat landing shall be provided at each wellhead platform for access by supply boat. The boat landing is modeled at the east side of the jacket and the boat landing loads are divided into four points, two mid points and two ends points of east side legs. The boat-landing load is 18T on each point, which will be described in weight and loads section.

2.2.3.2 Caissons

- One open drain caisson 914mm (36") diameter with 15 mm thickness
- One seawater caisson 914mm (36") diameter with 15 mm thickness

2.2.3.3 Risers

- One 446.6mm (18") diameter with 12 mm thickness
- One 114mm (4") diameter with 12 mm thickness

2.2.3.4 J-tubes

- Two 273mm (10") for fiber optic cable with 12 mm thickness

2.3 Foundation

A pile-soil interaction program shall be used to model the pile foundation comprising the pile from mud line to the target penetration and the associated non-linear lateral p-y, axial t-z, and end bearing q-z soil response curves. This soil design data, which is used in the foundation analyses, is given in Fugro geotechnical report [7].

Equivalent linear pile super element containing the foundation stiffness associated with the respective loading condition will be used for earthquake analysis. Foundation design shall be consistent with API RP2A guidelines. The pile shall be designed for the following design load conditions;

- Operating phase
- Boat impact
- Fatigue

For the pile ultimate capacity, refer to Fugro final report No. PP97. The ultimate vertical capacity of the piles are presented as Ultimate Capacity Curves in following which will be used to determine the target penetration with API RP 2A 6.3.4 factors safety of 1.5 for the extreme storm and boat impact and 2.0 for the operating storm

Table 2.3 : Summary of Soil Description at SPD25 Location

Soil Description	Top of Unit below seabed (m)	Thickness (m)
Very soft to firm , sandy, siliceous carbonate SILT with coarse coral gravels	0.0	1.9
Fine to coarse, clayey to very clayey, siliceous carbonate SAND with cemented shells and corals	1.85-2.00	3.4
Soft to stiff, locally slightly cemented, carbonate CLAY, with shells	5.00-5.55	2.00
Medium to coarse, slightly cemented, silty, siliceous carbonate SAND, with many shells	7.0-7.50	4.70
Soft to firm, sandy, carbonate CLAY	11.70-12.2	4.40
Firm to stiff, calcareous CLAY	16.20-16.50	13.40
Stiff to hard, carbonate CLAY, locally with few silt pockets	29.50-30.00	5.40
Fine to coarse, slightly cemented, very clayey, siliceous carbonate SAND, with shells	34.85-35.50	2.70
Very stiff becoming hard, locally slightly cemented, sandy, carbonates CLAY with shells between 48.10 m and 50.4m: lens of slightly cemented, very silty, carbonate SAND	37.75-38.00	38.60
Very weak, siliceous CALCARENITE	76.5- (>80.50)	0.50
Hard, greenish grey, carbonate CLAY	77.0	13.40
Dark grey to light grey, very gravely, very sandy, carbonate CLAY	90.4	2.60
Hard, greenish grey, carbonate CLAY	93.0	>17.5

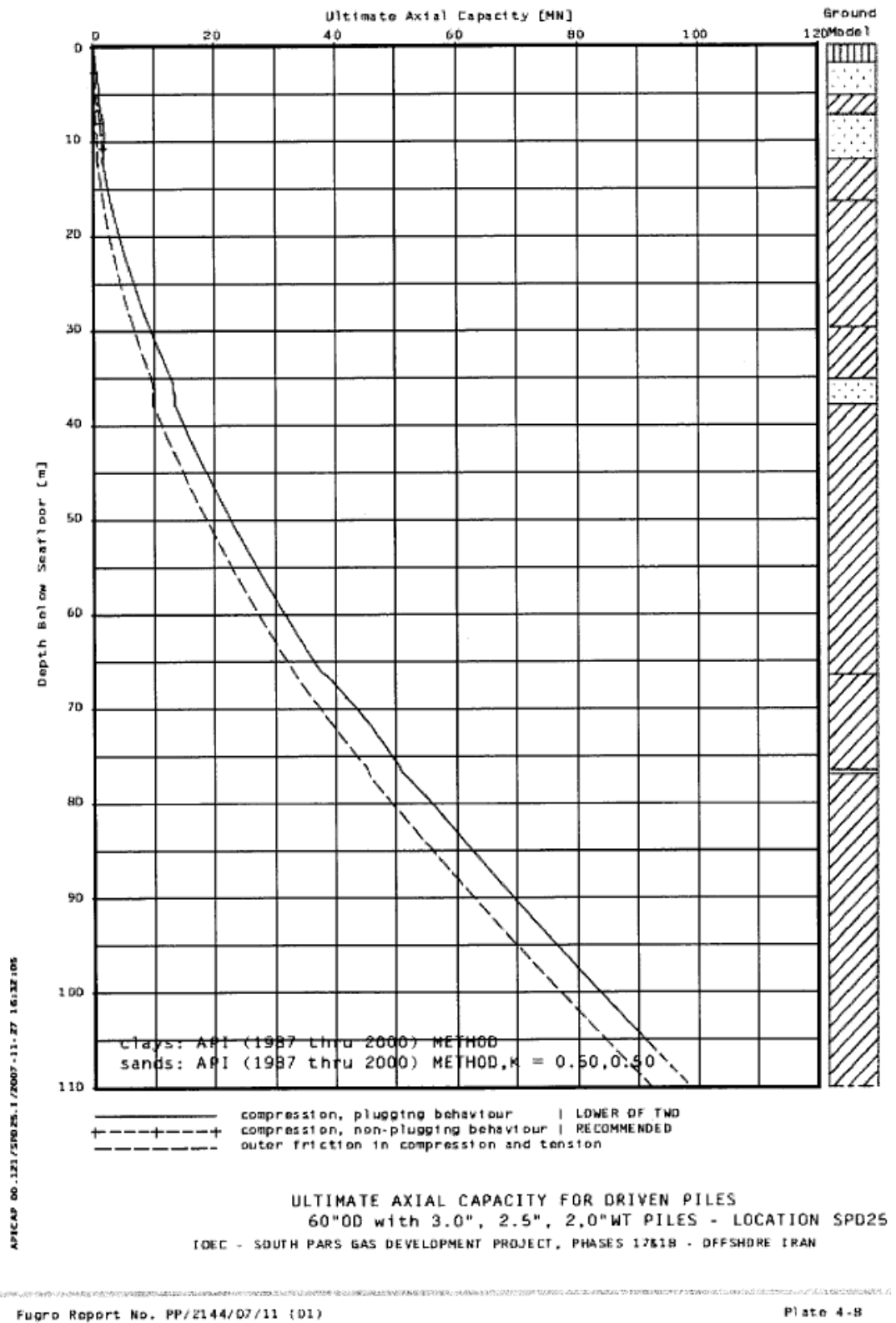


Figure 2.7 : 60 inch Pile Ultimate Capacity at SPD25 Location

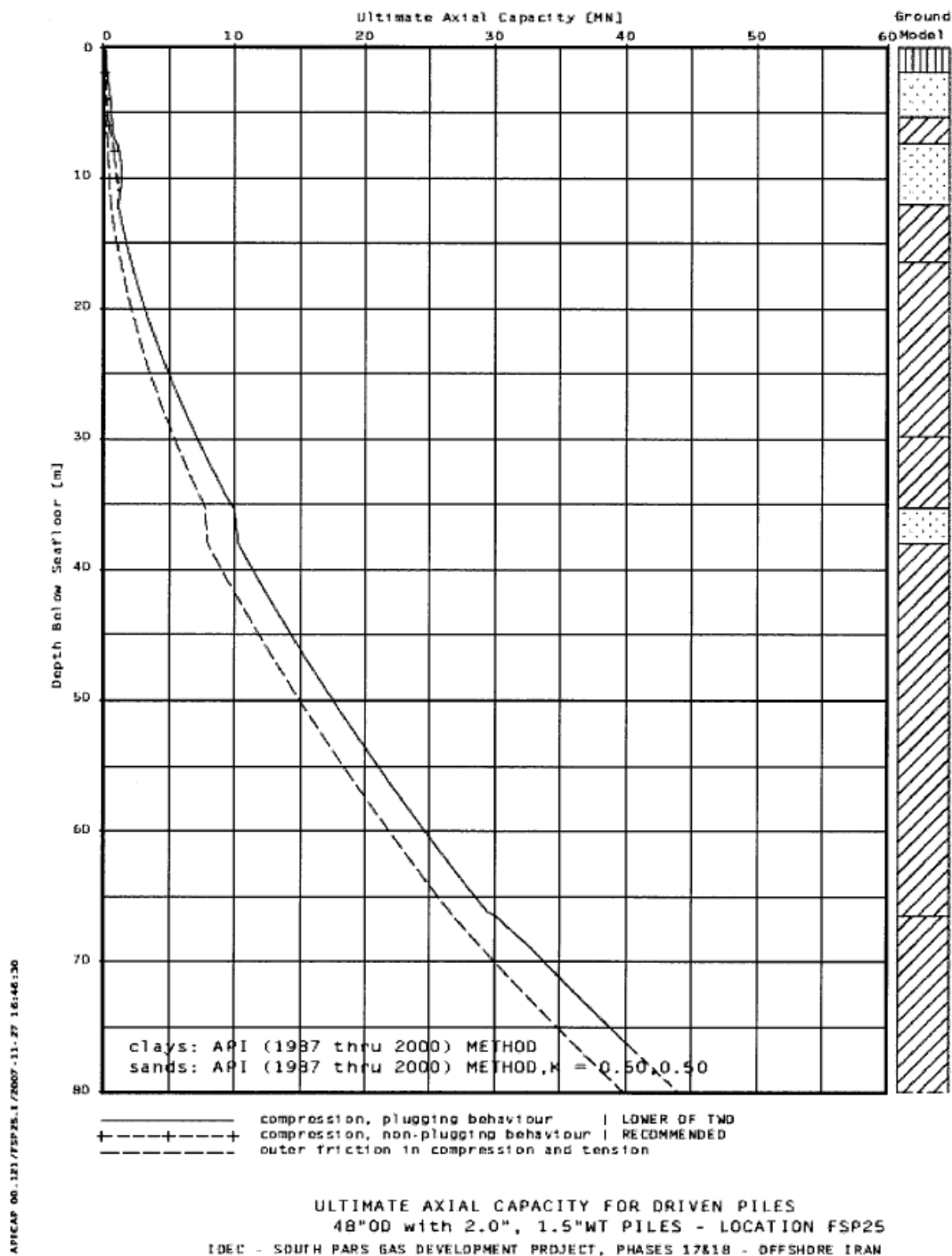
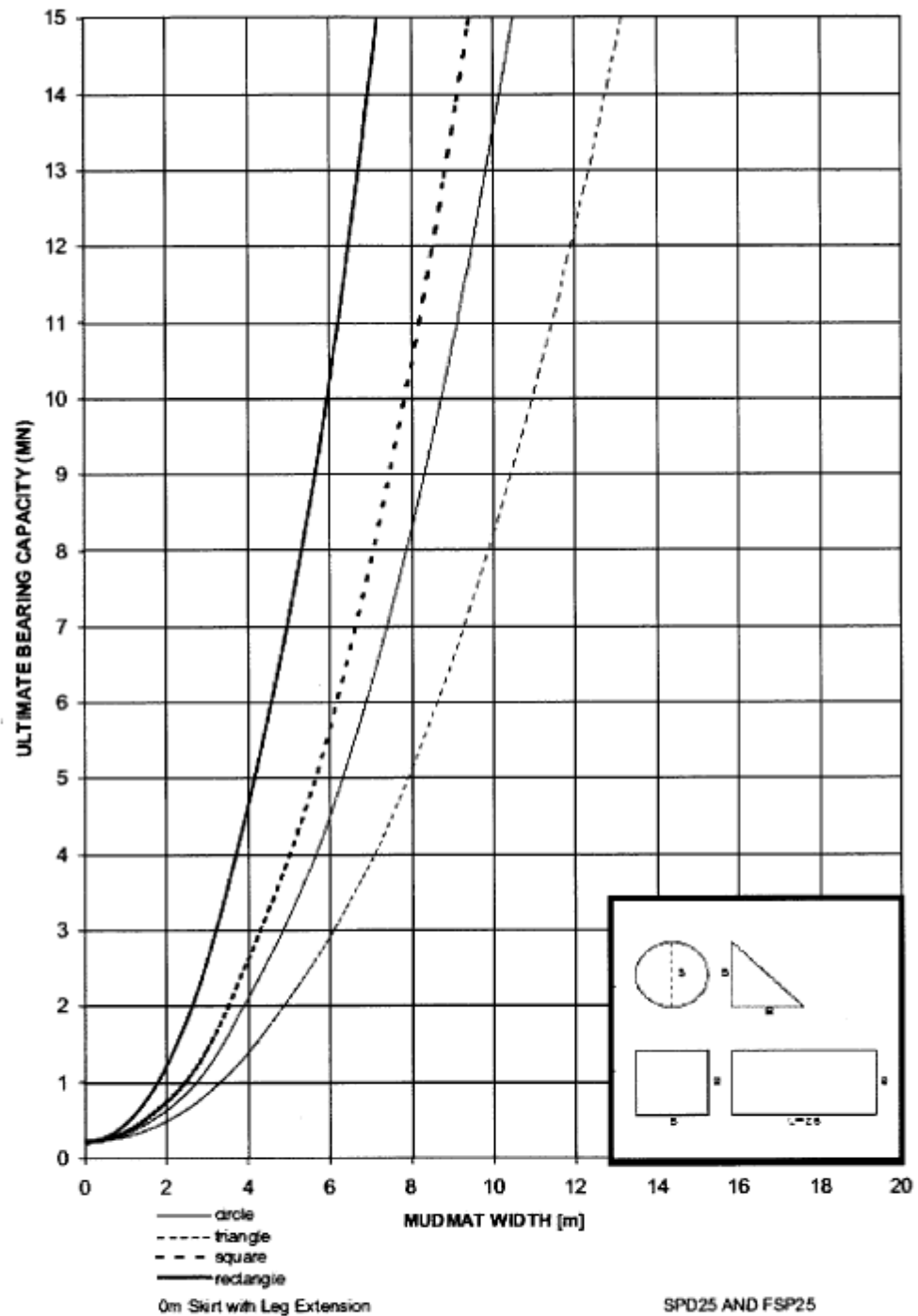


Figure 2.8 : 48 inch Pile Ultimate Capacity at SPD25 Location



MUDMAT ULTIMATE BEARING CAPACITY
 IOEC - SOUTH PARS GAS DEVELOPMENT PROJECT, PHASE 17 & 18, OFFSHORE IRAN

Figure 2.9 : Mud mat Bearing Capacity at SPD25 Location

2.4 Weight Data and Design Loads

The jacket structures shall be designed for the pre-service and the in-service conditions. In this thesis, the used ET seismic analysis method is the sub root of in-place seismic condition. The earthquake load should be combined with other simultaneous loading such as gravity, buoyancy, and hydrostatic pressure. Gravity loading should include the platform dead weight (comprised of the weight of the structure, equipment, and appurtenances), live loads according to the API RP2A 2.3.6.c. Therefore, the mentioned loads are explicated in this section and modeled in the computer, but the other loads such as wind, wave, temperature etc are neglected.

2.4.1 Substructures Mass

The tables below tabulate a breakdown of masses of the topside and substructure. Mass of steel, bridge, jacket appurtenance, crane, helideck, and live load were modeled in accordance to the respective location.

The masses that are generated automatically by the SAP Software include:

- Jacket members
- Conductors (including casing and grout)
- Risers
- J-tubes
- Boat landing; is manually input into related specification in the SAP model.
- Marine growth; is generated by the SAP.
- Added mass; added mass coefficient of 1.07 to account for sacrificial anodes

The weight of non-modeled jacket appurtenances with consideration of a contingency factor of 1.10 is in table 2.4 and the Center Of Gravity (COG) during SPD25 Jacket operating is in Figure 2.9. The non-modeled jacket appurtenance 2930.32KN weight was manually input at the jacket COG. [10]

Table 2.4 : Jacket Appurtenance Weight

Description	Base Weight (KN)	Contingency Factor	Factored Weight (KN)
Sea deck Walkway	-212.39	1.10	-233.63
Launch Cradles	-612.42	1.10	-673.66
Launch Truss Ring Stiffener	-90.48	1.10	-99.53
Launch Cradles (Buoyancy)	205.85	1.10	226.44
Anodes	-866.23	1.10	-952.85
Anodes (Buoyancy)	327	1.10	359.70
Flooding and Grouting Lines	-67.65	1.10	-74.42
Buoyancy Tank Supports	-49.04	1.10	-53.94
Hydrostatic Collapse Rings	-76.19	1.10	-83.81
Pile Spacers	-30.8	1.10	-33.88
Upending Pad eyes	-55.64	1.10	-61.20
Conductor Guides/Cones	-268.44	1.10	-295.28
Crown Shim Plates	-37.08	1.10	-40.79
Load out Pulling Eyes	-19.45	1.10	-21.40
Launch Lugs	-11.92	1.10	-13.11
20" Riser Dead Weight Support Clamps	-34.19	1.10	-37.61
20" Riser Guide Clamps	-9.32	1.10	-10.25
4" Riser Dead Weight Support Clamps	-8.72	1.10	-9.59
4" Fuser Sliding Clamps	-2.39	1.10	-2.63
Boat landing Appurtenances Weight	-277.99	1.10	-305.79
Pile Stabbing Guide	-384.55	1.10	-423.01
Flooding & Ballasting System	-40.37	1.10	-44.41
Launch Lug Tubular 508 OD	-41.52	1.10	-45.67
Total	-2663.93		-2930.32

The jacket center of gravity is:

X= -2.154 Y= -0.343 Z= 14.599

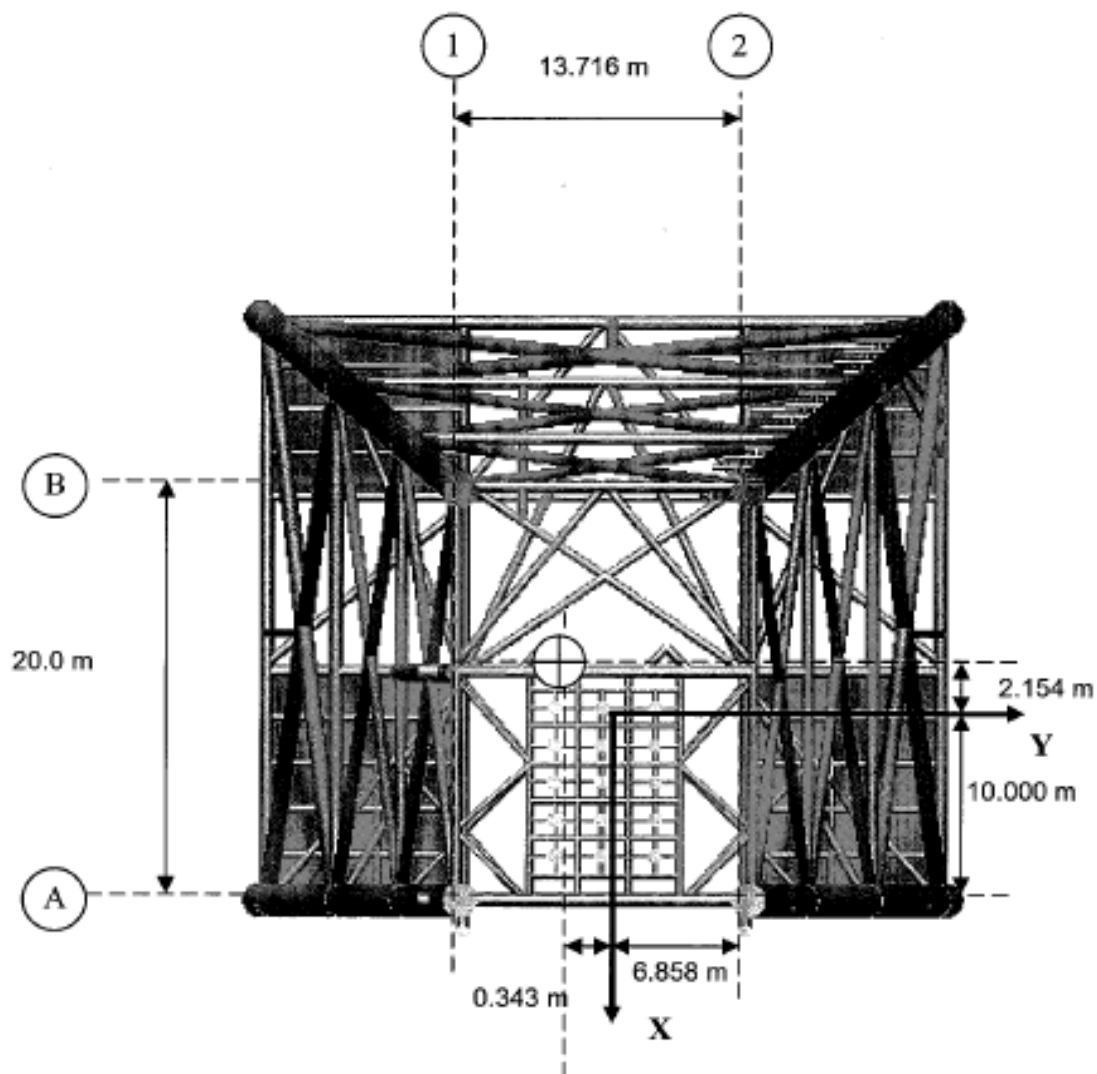


Figure 2.10 : SPD25 Jacket Operating COG Plan View

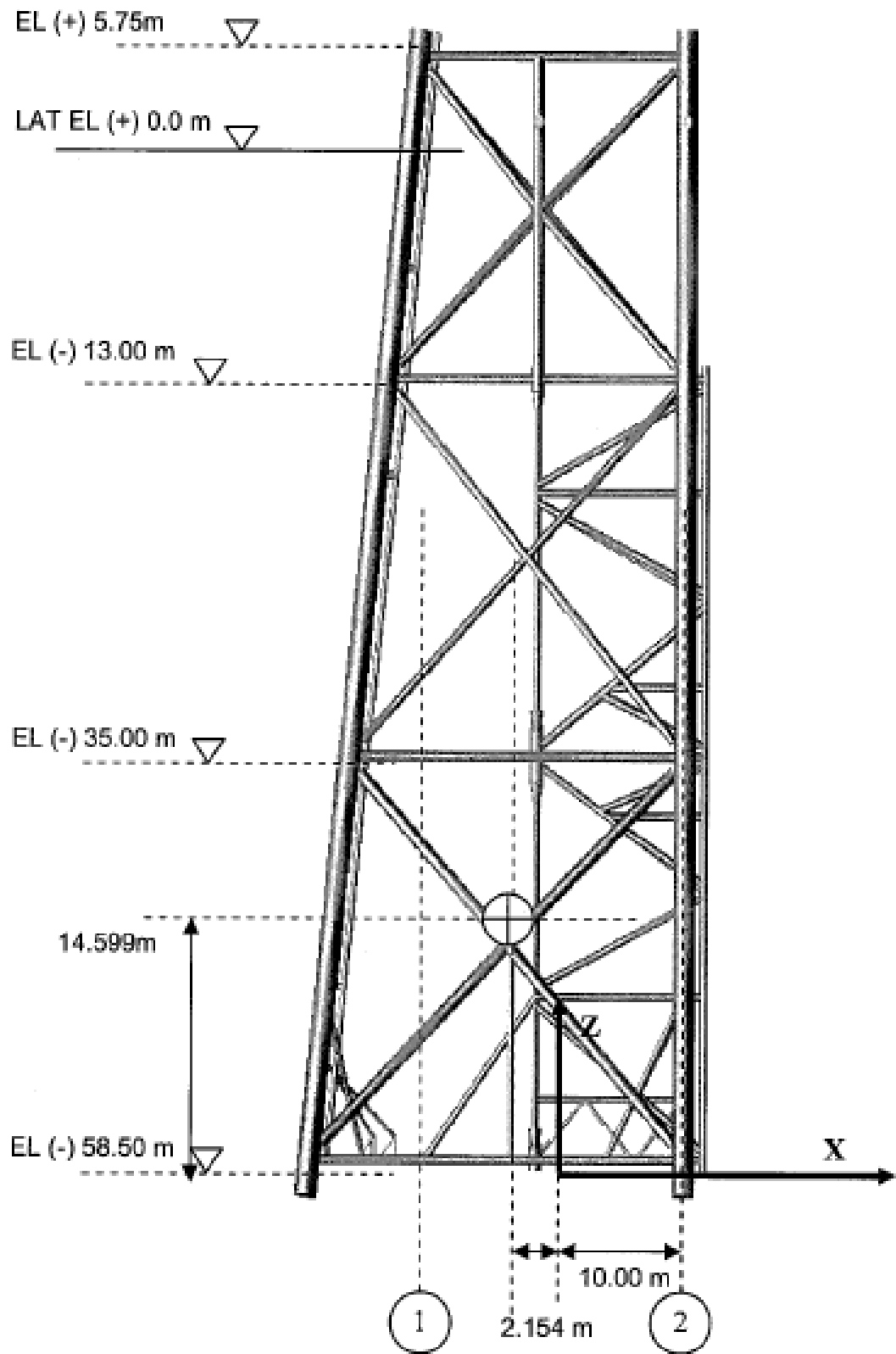


Figure 2.11 : SPD25 Jacket Operating COG Elevation

2.4.2 Topside mass

The topside masses are converted from in-place operating loads and it comprises;

- Steel (primary, secondary and tertiary)
- Equipment operating
- Piping operating
- Electrical and instrument
- Area live loads

The operating weight of the SPD25 is 2820T (75% storage load), 151T bridge load, 24T crane operating axial load, 72T boat landing and 13000 lb (58KN) helideck which explicated in the below tables. [10]

The maximum 2971.24T (2820T + 151T) operating mass equal to 29147.86KN weight is modeled at the topside center of gravity and connected to the pile head with four dummy rigid members. Dummy rigid members are members, which have no mass but stiffness to convert the masses to the pile heads.

The following tables are breakdown of the final discipline weights and masses with consideration of a contingency factor of 1.05, from Weight Control Report rev2 for SPD25 Wellhead Platform Deck.

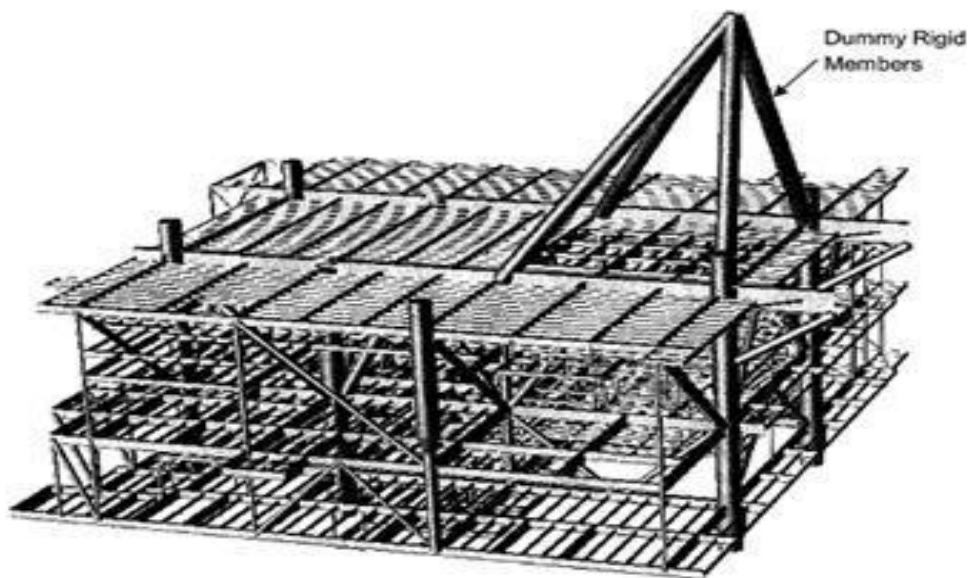


Figure 2.12 : Dummy Rigid Members

Table 2.5 : Topside Generated Mass Breakdown

Description	Mass (T)
Deck Primary Steel Self Weight	764.35
Tertiary Steel at Helideck	135.08
Tertiary Steel at Upper Deck	67.58
Tertiary Steel at Lower Deck	99.01
Tertiary Steel at Cellar Deck	105.88
Tertiary Steel At Drain Deck	13.37
Tertiary Steel at Wellhead Service Platform	8.62
Tertiary Steel at Star Tower 1 & 2	23.71
Tertiary Steel at Service Platforms	7.13
Equipment Dry Weight	347.81
Equipment Content Weight	255.70
Piping Dry Weight	477.80
Piping Content Weight	66.59
Instrument Dry Weight	110.49
Electrical Dry Weight	54.38
Safety Equipment Dry Weight	36.20
Safely Equipment Content Weight	19.80
Architectural Dry Weight	40.49
Bridge Gravity Reaction for Operating Condition	151.04
Crane Dead Load(Operating Condition)	45.25
Unoccupied Deck Area Live Load	109.92
Lay down Area Live Load (10KPa)	31.04
Maximum Topside Operating Mass	2971.24

Table 2.6 : Topside Basic Load Cases Weight Breakdown

Description	Base Weight (KN)	Contingency Factor	Factored Weight (KN)
Deck Primary Steel Self Weight	-7141.17	1.05	-7498.23
Tertiary Steel at Helideck	-1232.68	1.075	-1325.13
Tertiary Steel At Upper Deck	-616.67	1.075	-662.92
Tertiary Steel At Low Deck	-903.54	1.075	-971.31
Tertiary Steel At Cellar Deck	-966.24	1.075	-1038.71
Tertiary Stool At Drain Deck	-122.01	1.075	-131.16
Tertiary Steel At Wellhead Service Platform	-78.62	1.075	-84.52
Tertiary Steel At Stair Tow 1 & 2	-216.41	1.075	-232.64
Tertiary Stool At Service Platforms	-65.02	1.075	-69.90
Equipment Dry Weight	-2966.95	1.15	-3411.99
Equipment Content Weight	-2181.22	1.15	-2508.40
Piping Dry Weight	-4261.11	1.1	-4687.22
Piping Content Weight	-593.89	1.1	-653.28
Instrument Dry Weight	-942.5	1.15	-1083.88
Electrical Dry Weight	-463.92	1.15	-533.51
Safety Equipment Dry Weight	-308.84	1.15	-355.17
Safety Equipment Content Weight	-168.9	1.15	-194.24
Architectural Dry Weight	-361.13	1.1	-397.24
Bridge Gravity Reaction For Operating Condition	-1481.74	1	-1481.74
Crane Operating Moment Mx	4081	0.75	3061.00
Crane Operating Moment My	-4081	0.75	-3061.00
Crane Dead Load (Operating Condition)	-386	1.15	-443.90
Unoccupied Deck Area Live Load	-1437.72	0.75	-1078.29
Lay down Area Live Load (10KPa)	-406	0.75	-304.50
Maximum Topside Operating Weight	-27302.28		-29147.86

The topside Center Of Gravity (COG) is:

$X = 2.567$ $Y = -0.073$ $Z = 70.946$

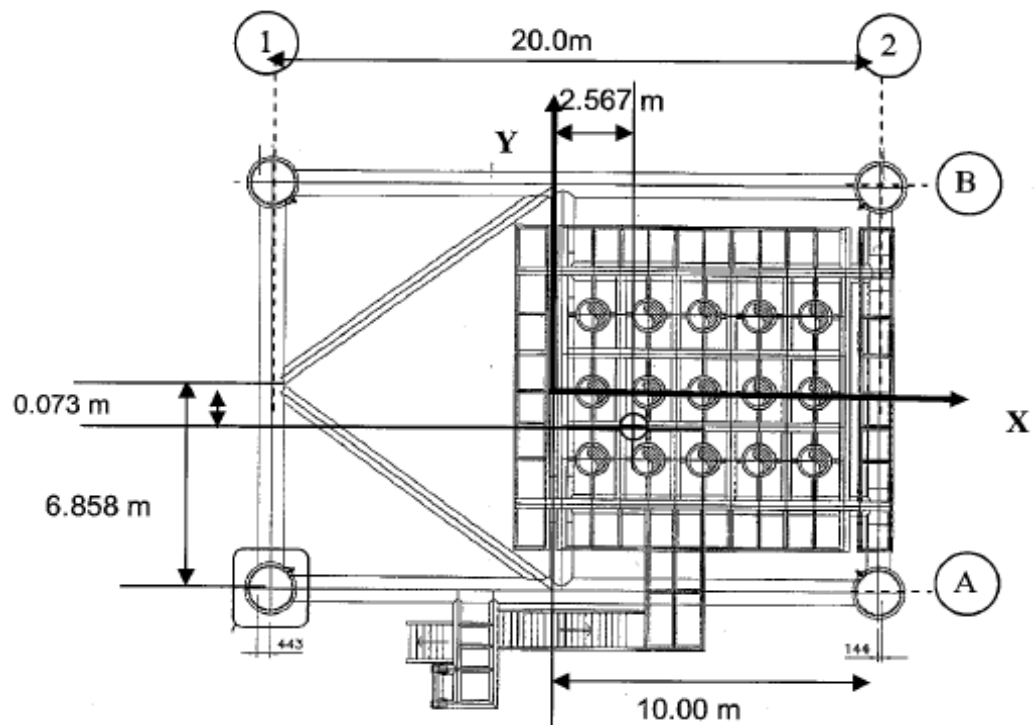


Figure 2.13 : SPD25 Topside Operating COG Plan View

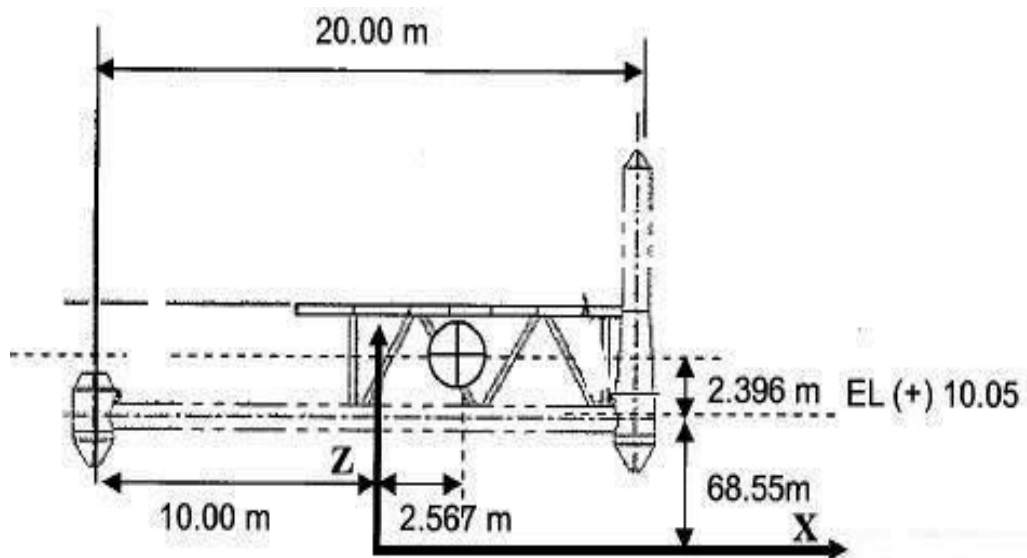


Figure 2.14 : SPD25 Topside Operating COG Elevation View

2.4.3 Crane Loads

Crane axial operating loads of 24T (75% of operating live load was used) is modeled at the center of crane mass on Row A, Column 1 & 2 of jacket (X= 21.42m, Y=23.65m, Z= 12.45m). A crane with a main hoist lifting capacity of 20T at 13m and 6T at 22m, and auxiliary hoist 5T at 22m, is on the east side of the platform. The crane dead load during operation is in the below table. The crane operating moments (Mx and My) are considered in the topside 2820T weight and modeled at the topside COG. [10]

Table 2.7 : Crane Loads

Description	Based Weight (KN)	Weight Contingency	Factored Weight (KN)
Crane Operating Axial Load	-314.0	0.75	-235.5
Crane Operating Moment, Mx	Mx= 3061 KNm		
Crane Operating Moment, My	My= 3061 KNm		

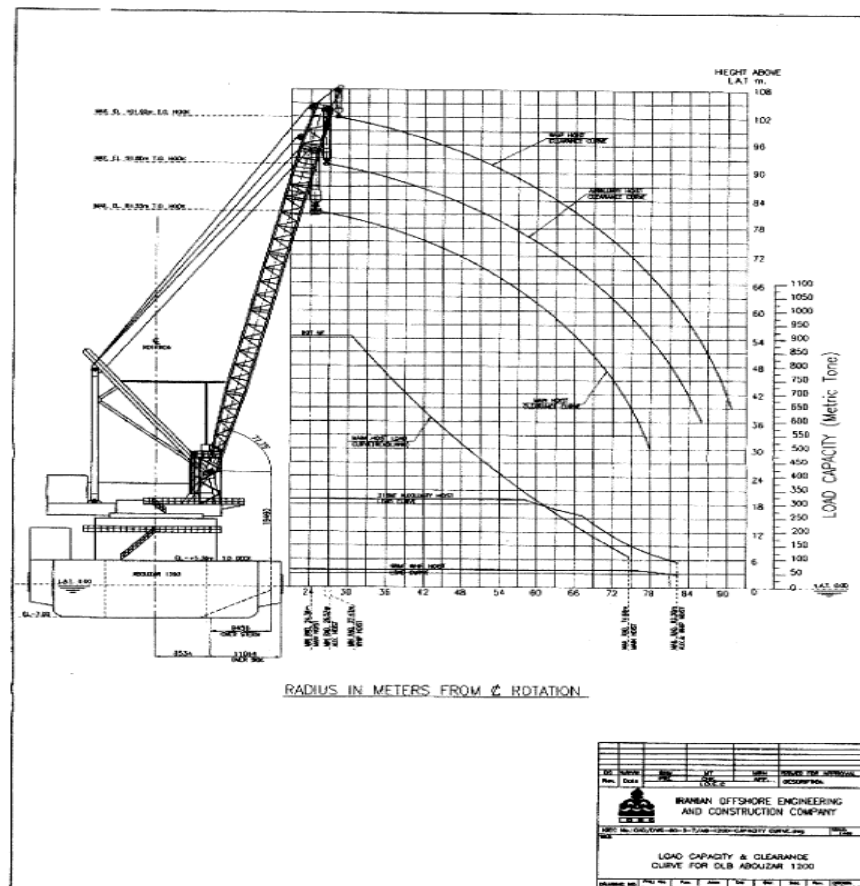


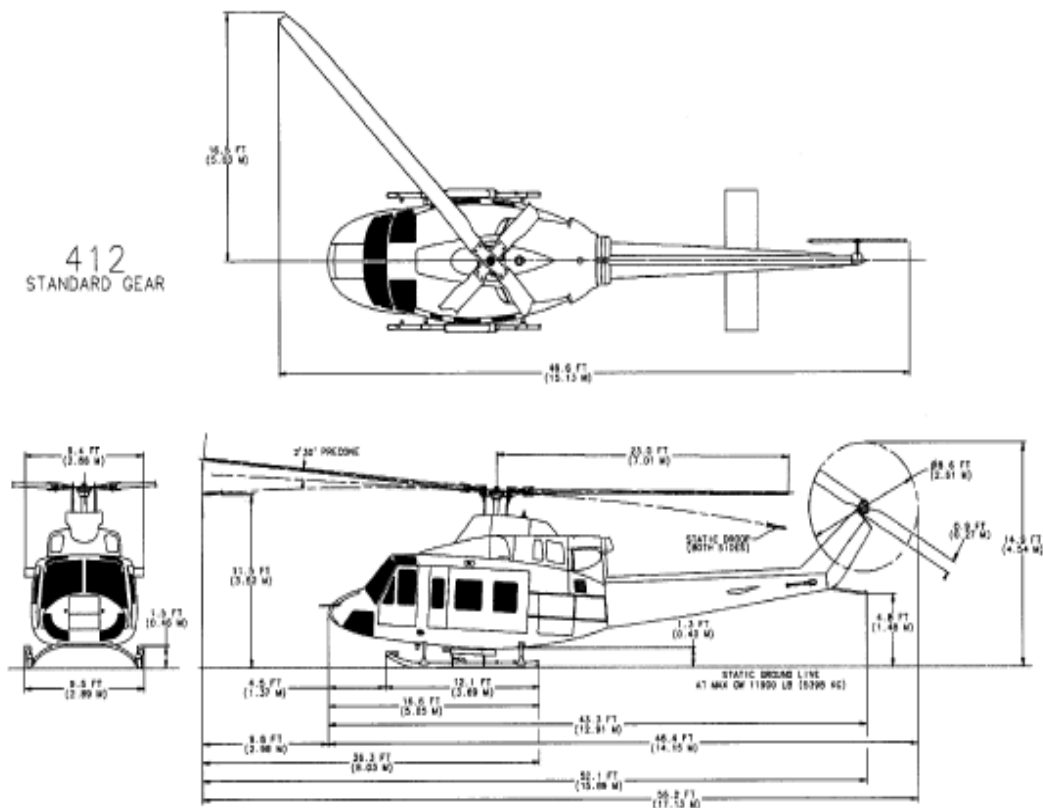
Figure 2.15 : Crane DLB Abouzar 1200

2.4.4 Helideck Weight

Helideck shall be designed for AGUSTA BELL AB 412 helicopter class type carrying 15 people. The gross weight is 13000 lbs, which is modeled at the helideck platform center of gravity (X= 15.82, Y= 16.43, Z= 12.45). [10]

Bell 412 Helicopter Design Data and Dimensions

Bell Helicopter
A Textron Company



LANDING GEAR LOADING AT MAXIMUM GROSS WEIGHT (11,900-POUNDS),
BASED ON 1G STATIC CONDITIONS AT AFT-MOST STRUCTURAL CG LIMIT

Gear Type	Loading (lb)		Contact Area (in ²)		Contract Pressure (lb/in ²)	
	Forward	Aft	Forward	Aft	Forward	Aft
Standard Skid	2356	9544	24.0 x 2.0	24.0 x 2.0	49	199
High Skid	2386	9514	24.0 x 2.0	24.0 x 2.0	50	199
Emergency Floats	2486	9514	24.0 x 2.0	24.0 x 2.0	50	199

Figure 2.16 : BELL Helicopter

2.4.5 Bridge Load

The bridge loads associated with the 100-year storm during operating condition is -1481.74KN in the Z direction. The bridge load is modeled at the topside COG. [10]

2.4.6 Boat Landing Load

One boat landing shall be provided at each wellhead platform for access by supply boat. The boat landing is modeled at the east side of the jacket and the 72T load is divided to four into four points, two mid points and two end points of east side legs (Columns 1 & 2, Row A).

{X= 10.445, Y= 8.279, Z= -0.74}, {X= 9.843, Y= 7.591, Z= -5.558} and

{X= 31.575, Y= 8.279, Z= -0.74}, {X= 31.575, Y= 7.591, Z= -5.558}

2.4.7 Area Live Loads

Area live loads at each deck are shown in table 2.8. Seventy-five percent (75%) of area live loads was used in the Seismic Analysis [API RP2A WSD 2.3.6c] [9]. The 1078.29KN unoccupied deck area live load is considered separately in the top side loads. The sum of live loads is applied in the top side COG.

Table 2.8 : Area Live Loads

Description	Area Loading (KN/m²)	Total Loading (KN)	75% × Total Loading (KN)
Helideck Lay down Area Live Load	-10.0	-82.50	-61.88
Upper Deck Laydown Area Live Load	-10.0	-148.00	-111.00
Lower Deck Laydown Area Live Load	-10.0	-78.00	-58.50
Cellar Deck Laydown Area Live Load	-10.0	-97.50	-73.13
Total		-406.00	-304.51

2.4.8 Marine Growth

Marine growth on the jacket will be included in the automatic mass generation and as lumped masses. The marine growth saturated density is 1400 kg/m³ and the elevation and the thickness are as the table below [10]:

Table 2.9: Marine Growth Profile

Elevation w.r.t. LAT (m)	Marine Growth Thickness (mm)
EL(-) 2.0 to EL(-) 6.0	75
EL(-) 6.0 to Mud line	50

2.4.9 Wave and Current Load

Two dimensional water practical velocities and accelerations will be calculated with Stokes 5th order wave theory and the Morison's equation to calculate drag and inertia forces on each element in the hydrodynamic model. [10]

The wave kinematic factor shall be 0.95 for the extreme storm waves and 1.0 for operating wave load calculations in according with API RP2A 2.3.1.b-3. [9]

Wave and current loads will be factored by appropriate dynamic amplification factors (DAF) based on the single degree of freedom DAF formula, the platform's natural period and wave period. The DAF of 1.1 will be applied as a hydrodynamic mass coefficient in the SAP model. [10]

$$DAF = \frac{1}{\sqrt{\left(1 - \left(\frac{T_s}{T_w}\right)^2\right)^2 + 4 \times \left(C \times \frac{T_s}{T_w}\right)^2}} \quad (2.1)$$

Where: T_s = Natural period of the structure

T_w = Wave period

C = Viscous damping coefficient = 0.020

The 1-year, 10-year and 100-year directional design wave height and wave period are in tables 2.10, 2.11 and 2.12. [10]

The 1-year and 100-year tide are in the table 2.13 and 2.4. [10]

Table 2.10 :1-year Directional Maximum and Significant Storm Tide and Wave

1 YEAR DIRECTIONAL MAXIMUM AND SIGNIFICANT STORM TIDE AND WAVE CHARACTERISTICS¹:
APPROXIMATELY 26°46'20"N, 52°05'59"E; SOUTH PARS GAS FIELD; PERSIAN GULF;
60 METER CHART DEPTH; ANNUAL

	<u>North</u>		<u>Northeast</u>		<u>East</u>		<u>Direction Southeast</u>		<u>South</u>		<u>Southwest</u>		<u>West</u>		<u>Northwest</u>	
	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>
Chart Depth, m.	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
Highest Astro- nomical Tide, m.	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9
Storm Tide, m.	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
Total Tide, m.	2.0	2.0	2.0	2.0	2.0	2.0	2.1	2.1	2.0	2.0	2.0	2.0	2.0	2.0	2.1	2.1
Still Water Depth, m.	62.0	62.0	62.0	62.0	62.0	62.0	62.1	62.1	62.0	62.0	62.0	62.0	62.0	62.0	62.1	62.1
Wave Height, m.	5.5	3.0	5.1	2.7	6.0	3.2	6.3	3.4	5.6	3.0	5.0	2.7	6.0	3.2	6.7	3.6
Wave Period, sec.	7.0	7.0	7.3	6.7	8.1	7.3	8.3	7.5	7.9	7.1	7.4	6.7	8.1	7.3	8.6	7.7
Spectral Peak Period, sec.		8.0		7.6		8.3		8.5		8.1		7.6		8.3		8.8
Crest Elevation Above Still Water Level, m.	3.0	1.6	2.8	1.4	3.3	1.7	3.5	1.8	3.1	1.6	2.7	1.4	3.3	1.7	3.6	1.9
Crest Elevation Above Chart Datum, m.	5.0	3.6	4.8	3.4	5.3	3.7	5.6	3.9	5.1	3.6	4.7	3.4	5.3	3.7	5.7	4.0
Crest Elevation Above Bottom, m.	65.0	63.6	64.0	63.4	65.3	63.7	65.6	63.9	65.1	63.6	64.7	63.4	65.3	63.7	65.7	64.0
Wave Length, m.	97.7	77.5	90.3	70.9	105.4	84.3	110.6	89.0	100.1	79.7	87.9	71.0	105.4	84.3	118.5	93.7

Notes: ¹Directions from which waves approach.

Table 2.11: 10-year Directional Maximum and Significant Storm Tide and Wave

10 YEAR DIRECTIONAL MAXIMUM AND SIGNIFICANT STORM TIDE AND WAVE CHARACTERISTICS¹:
APPROXIMATELY 26°46'20"N, 52°05'59"E; SOUTH PARS GAS FIELD; PERSIAN GULF;
60 METER CHART DEPTH; ANNUAL

	<u>North</u>		<u>Northeast</u>		<u>East</u>		<u>Direction Southeast</u>		<u>South</u>		<u>Southwest</u>		<u>West</u>		<u>Northwest</u>	
	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>
Chart Depth, m.	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
Highest Astro- nomical Tide, m.	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9
Storm Tide, m.	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.2	0.2	0.2	0.2
Total Tide, m.	2.0	2.0	2.0	2.0	2.1	2.1	2.1	2.1	2.1	2.1	2.0	2.0	2.1	2.1	2.1	2.1
Still Water Depth, m.	62.0	62.0	62.0	62.0	62.1	62.1	62.1	62.1	62.1	62.1	62.0	62.0	62.1	62.1	62.1	62.1
Wave Height, m.	7.6	4.1	6.9	3.7	8.4	4.3	9.0	4.8	8.0	4.3	6.9	3.7	8.4	4.3	9.4	5.1
Wave Period, sec.	9.0	8.1	8.7	7.8	9.4	8.3	9.7	8.7	9.2	8.3	8.7	7.8	9.4	8.3	9.9	8.9
Spectral Peak Period, sec.		9.2		8.9		9.7		9.9		9.4		8.9		9.7		10.1
Crest Elevation Above Still Water Level, m.	4.2	2.2	3.8	2.0	4.6	2.4	4.9	2.6	4.4	2.3	3.8	2.0	4.6	2.4	5.2	2.7
Crest Elevation Above Chart Datum, m.	6.2	4.2	5.8	4.0	6.7	4.5	7.0	4.7	6.5	4.4	5.8	4.0	6.7	4.5	7.3	4.8
Crest Elevation Above Bottom, m.	66.2	64.2	65.8	64.0	66.7	64.5	67.0	64.7	66.5	64.4	65.8	64.0	66.7	64.5	67.3	64.8
Wave Length, m.	129.7	103.0	121.3	96.3	141.0	114.3	150.0	119.5	135.2	109.0	121.3	96.3	141.3	114.3	155.0	129.1

Notes: ¹Directions from which waves approach.

Table 2.12: 100-year Directional Maximum and Significant Storm Tide and Wave

**100 YEAR DIRECTIONAL MAXIMUM AND SIGNIFICANT STORM TIDE AND WAVE CHARACTERISTICS¹:
APPROXIMATELY 26°46'20"N, 52°05'59"E; SOUTH PARS GAS FIELD; PERSIAN GULF;
60 METER CHART DEPTH; ANNUAL**

	<u>North</u>		<u>Northeast</u>		<u>East</u>		<u>Direction Southeast</u>		<u>South</u>		<u>Southwest</u>		<u>West</u>		<u>Northwest</u>	
	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>	<u>Max</u>	<u>Sig</u>
Chart Depth, m.	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
Highest Astronomical Tide, m.	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9
Storm Tide, m.	0.1	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.2	0.2	0.1	0.1	0.2	0.2	0.3	0.3
Total Tide, m.	2.0	2.0	2.0	2.0	2.1	2.1	2.2	2.2	2.1	2.1	2.0	2.0	2.1	2.1	2.2	2.2
Still Water Depth, m.	62.0	62.0	62.0	62.0	62.1	62.1	62.2	62.2	62.1	62.1	62.0	62.0	62.1	62.1	62.2	62.2
Wave Height, m.	9.7	5.2	8.8	4.8	10.8	5.8	11.6	6.2	10.2	5.5	8.8	4.7	10.0	5.8	12.2	6.6
Wave Period, sec.	10.0	9.0	9.6	8.6	10.4	9.4	10.8	9.8	10.2	9.2	9.5	8.5	10.4	9.4	11.0	10.0
Spectral Peak Period, sec.		10.2		9.0		10.7		11.1		10.5		9.7		10.7		11.4
Crest Elevation Above Still Water Level, m.	5.4	2.8	4.9	2.5	6.0	3.1	6.4	3.3	5.7	2.9	4.0	2.5	6.0	3.1	6.0	3.5
Crest Elevation Above Chart Datum, m.	7.4	4.8	6.9	4.5	8.1	5.2	8.6	5.5	7.0	5.0	6.8	4.5	8.1	5.2	9.0	5.7
Crest Elevation Above Bottom, m.	67.4	64.8	66.9	64.5	68.1	65.2	68.6	65.5	67.8	65.0	66.8	64.5	68.1	65.2	69.0	65.7
Wave Length, m.	159.4	127.9	147.1	117.0	171.2	139.1	184.0	150.6	165.4	133.7	144.5	114.3	171.2	139.1	190.1	156.5

Note: ¹Directions from which waves approach.

Table 2.13 : 1-year Storm Tide and Design Water Depth

Platform	Water Depth (m) w.r.t. LAT	Storm Surge (m)	Highest Astronomical Tide (m)	Subsidence (m)	Max Water Depth (m)
SPD 25 WHP	58.50	0.2	1.9	0.3	60.9

Table 2.14 : 100-year Storm Tide and Design Water Depth

Platform	Water Depth (m) w.r.t. LAT	Storm Surge (m)	Highest Astronomical Tide (m)	Subsidence (m)	Max Water Depth (m)
SPD 25 WHP	58.50	0.3	1.9	0.3	61.0

2.4.10 Hydrostatic Pressure

The hydrostatic pressure is considered in the horizontal and diagonal members such as braces and beams. The water density is $\rho = 1.023 \text{ ton}/m^3$ as mentioned before. The hydrostatic pressure is obtained by multiplying the density by the member area, since this load is distributed, the force shall be divided to the member length. This load is applied to the horizontal and diagonal members under the seawater.

2.5 Member Section Properties

Jacket members are modeled joint to joint. Members are assigned to groups based on their member section and other properties. Member section properties and yield strength are shown in table below [10], [11]:

Table 2.15 : Member Section Properties

Item	Material	Yield (MPa)	Ultimate Tensile Strength (MPa)
Tubular $\phi > 20''$	EN 10025 S355J0	355 ($t \leq 25\text{mm}$)	490
	EN 10025 S355K2	345 ($25 < t \leq 50\text{mm}$)	
	EN 10025 S355NL	355 ($50 < t \leq 70\text{mm}$)	
	EN 10025 S355NL	325 ($70 < t \leq 100\text{mm}$)	
Tubular $\phi \leq 20''$	API 5L X52 SR4/SR5	360	455
Joint Can Thickness $> 25\text{mm}$	EN 10025 S355 Z35	345 ($25 < t \leq 50\text{mm}$)	490
	EN 10025 S355 Z35	355 ($50 < t \leq 70\text{mm}$)	490
Joint Can Thickness $\leq 20\text{mm}$	API 5L X52 SR4/SR5	360 ($t \leq 20\text{mm}$)	455

Whereas the jacket design is based on the member stresses, the load combination factors are 1.0. So that, the members yield stresses can be increased up to $1.0F_y$. The design allowable stresses are as following:

Table 2.16 : Design Allowable Stresses

Tensile stress	$0.6 F_y$ (AISC D1)
Compressive stress	$0.6 F_y$ (AISC E2)
Bending stress	$0.66 F_y$ (AISC F2)
Shear stress	$0.4 F_y$ (AISC F4)
Bearing stress	$0.75 F_y$ (AISC J8)
F_y	Yield stress for steel material

The members bending moment amplification factor (C_m) is mentioned in table 1.1. The members D/T ratio range defined by API RP 2A 8.2.3 and 6.10.6 for piles and AISC are in table below.

Table 2.17 : D/T Ratio

Members	D/T Ratio	
	Maximum	Minimum
Brace	300	30
Joint cans	120	19
Piles	50	Note 1

Note 1: Minimum Pile wall thickness is $6.35 + D/100$ mm, where D is the diameter in mm.

3. FLOW FORECASTING

In this project, a new methodology for extending the application of Endurance Time method into the area of performance-based design is introduced. In fact, the characteristic of the ET method can best be utilized by extending its concepts to incorporate the basic concepts from performance-based design. The corresponding Performance Levels with time are defined for accelerogram functions. The mentioned modeled jacket is used in order to apply the above methods. And finally the Endurance Time method results are compared with Time history and pushover analysis.

3.1 Performance-Based Design Method

A real displacement, which will vary by the time passing, will occur in the structure when it is subjected to the seismic effects. Due to these seismic effects, the forces and stresses will be made in the structural members. The stresses are in elastic range in a low seismic motion and structure has no damage. Nevertheless, if these stresses were more than elastic range, kind of damage such as cracking, smashing, buckling or yield would occur in some members. Therefore, stresses could cause failure in some members and may cause total collapse of structure by increasing the seismic motion.

3.1.1 Performance Levels

Performance Levels are the ranges of structural response to a subjected seismic, which contain above damage types. The Performance Level Ranges and structural behavior under intensifying lateral drift are shown in Figure below.

The four discrete Structural Performance Levels are defined based on the FEMA standards:

- a) Operational (OP)
- b) Immediate Occupancy (IO)
- c) Life Safety (LS)
- d) Collapse Prevention (CP)

The IO level shall be defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified in FEMA356 for this Structural Performance Level. [12]

The LS level shall be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in standard for this Structural performance Level. [12]

The CP level shall be defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified in the FEMA356 for this Structural Performance Level. [12]

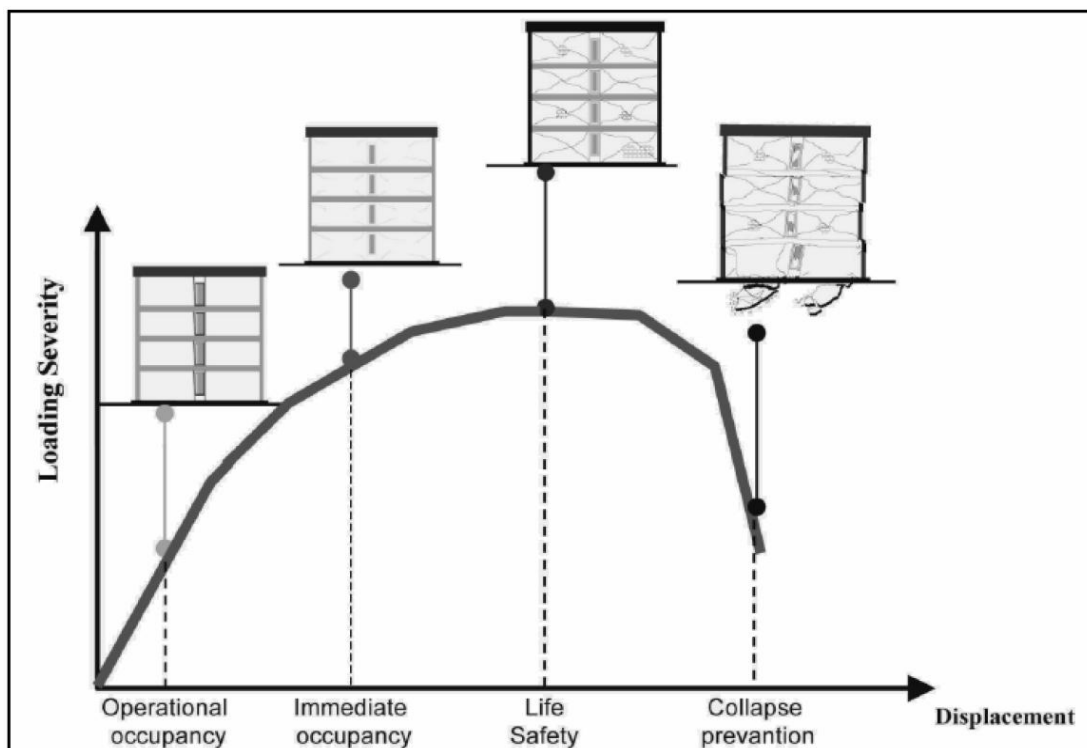


Figure 3.1 : Structural Performance Levels

There are two intermediate Structural Performance Ranges in addition to the mentioned three levels:

- a) Damage Control
- b) Limited Safety

The Damage Control level shall be defined as the continuous range of damage states between the LS Structural PL and the IO Structural PL. [12]. In this range, structure is in elastic mode generally.

The Limited Safety level shall be defined as the continuous range of damage states between the LS Structural PL and the CP Structural PL. [12]. In this range, structure has a considerable non-elastic behavior. Figure3.2

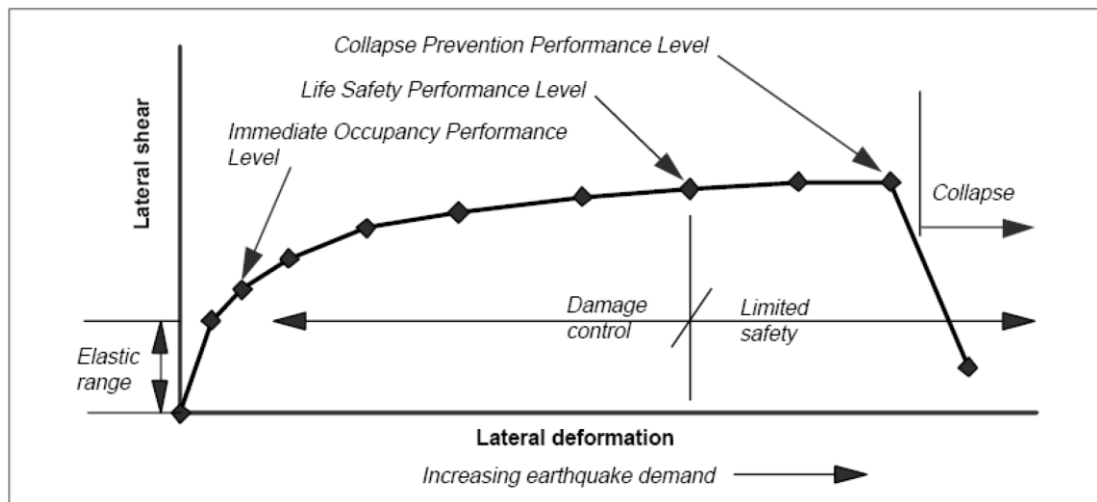


Figure 3.2 : Structural Performance Levels & Ranges

The Performance Levels are usually expressed as some distinct bands in the damage spectrum of a structure, and divide the damage status of structures according to the amount of damage to structural and nonstructural components. Moreover, some other concepts, such as cost, repair time and injury can be related to performance levels, too. [6]

Noting that the Performance Levels (PLs) are usually investigated at some specific levels of design earthquake motion, they can be thought of as a criteria for limiting values of measurable structural response parameters (such as story drift and absolute acceleration and maximum displacement) under each mentioned level of earthquake motion [6]. On the other hand, in FFMA-356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings, four levels of probabilistic earthquake hazard are defined [12].

Combining these levels with the PLs, a table of Performance Objectives (POs) can be created. These objectives are different according to the type of structure that is to be built. For example, if a hospital is planned to be built, an appropriate PO might be that it is capable of meeting the LS performance level in an earthquake with a mean return period of 2475 years, and the IO performance level in an earthquake with a mean return period of 475 years. [5]

Since the descriptions of the performance objectives are mostly qualitative, some performance criteria have been defined to bind these descriptions to engineering demand parameters, so that the performance objectives can be predicted in the analysis and design process. In fact, these criteria are the rules and guidelines that must be met to ensure that the designed structure satisfies the performance objectives. In this research, the performance level shown in Table 3.1 is considered for a residential building. [6]

Table 3.1 : Selected Performance Objective for a Structure

Earthquake Having Probability of Exceedance	Mean Return Periods (years)	Performance Level
50% per 50 year	75	IO
10% per 50 year	475	LS
2% per 50 year	2475	CP

The Performance Levels are defined by different parameters which introduced by the FEMA standards. In order to define Performance Levels, story drift ratio and member's plastic rotation parameters are considered in this thesis. Moreover, there are other parameters such as column compression force or compression and tensile forces of a brace, which made by earthquake, can be used to define performance levels. As we assumed the weak beam and strong column in this thesis, these parameters are not considered.

Definition of the Structural Performance depends on the different indices, which considered based on the structure types. The important indices are story drift ratio and member plastic rotation index. In order to control the structures conditions, the certain values of these indices are defined for each performance levels. Evaluating the relations of the Structural Performance which expressed by FEMA351, include some parameters which must be defined based on the related tables. There are certain tables for each mentioned indices in FEMA356, e.g. the below tables explicate story drift ratio and plastic rotation values. Story drift ratio of concrete and steel frame structures are shown in Table3.2.

The drift values given in Table3.2 are typical values provided to illustrate the overall structural response associated with various Structural Performance levels.

The plastic rotation ranges (for beams different strength modes against lateral bending) are shown in Table3.3. These values are defined according to the used substance. The general substance behavior is depicted in Figure3.3. [6]

Noting that in order to define Performance Level in each certain earthquake, the index that makes a critical condition must be choose as a determinant index. For instance, in one earthquake with certain PGA, member plastic rotation index may cause more critical condition but in the other earthquake with the different PGA, stories drift ratio may be determinant. [13]

Table 3.2 : Story Drifts Ratio Range Equal to Performance Levels for Different Frames [6]

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

Table 3.3 : Plastic Rotation Range Equal to Performance Levels for Beam & Column [6]

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components								
Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
				IO	Primary		Secondary	
	a	b	c		LS	CP	LS	CP
Beams—flexure								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	90 _y	110 _y	0.8	10 _y	60 _y	80 _y	90 _y	110 _y
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	40 _y	60 _y	0.2	0.250 _y	20 _y	30 _y	30 _y	40 _y
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Columns—flexure ^{2,7}								
For $P/P_{CL} < 0.20$								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	90 _y	110 _y	0.8	10 _y	60 _y	80 _y	90 _y	110 _y
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	40 _y	60 _y	0.2	0.250 _y	20 _y	30 _y	30 _y	40 _y
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

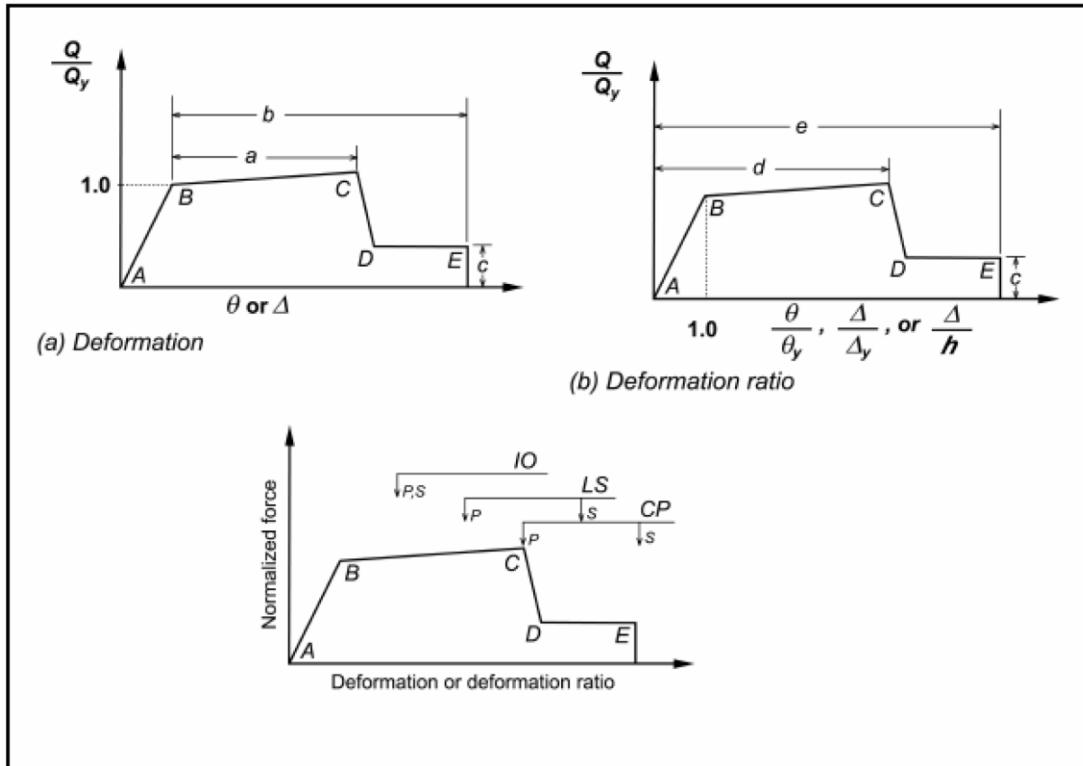


Figure 3.3 : Substance Plastic Behavior under Lateral Force [6]

3.1.2 Performance-Based Design Concept

Engineering base on the performance includes design, evaluation, construction, and maintenance of structures, which defined for a certain performance by client. To design base on the Performance Levels, reducing the imposed damaged to structure by earthquake and reducing the cost must always considered by engineers. [12]

On the other hand, the performance is defined according to the damage state after seismic occurred. There are two steps before starting designation based on the performance; First, definition of the structure type: it means that the structure is a hospital or ordinary building or the nuclear reactor. Second, definition of a user; it means that who may use the structure and what is expected from building. [14]

The brief performance-based design stages are as below:

- Defining the Performance Objectives
- Preliminary numerical design: the required characteristics of the structure such as maximum frequency, minimum required stiffness, critical loads combination etc would be obtained by considering the performance level and expected earthquakes and the properties of the structure such as structure system and geometry. Then, the structural members shall be designed based on the obtained values.
- Final design: after preliminary design, the structure imposed to the various loading and all the members investigated. If the structure were in expected performance range it is ok, if not, the preliminary design must be corrected. [5]

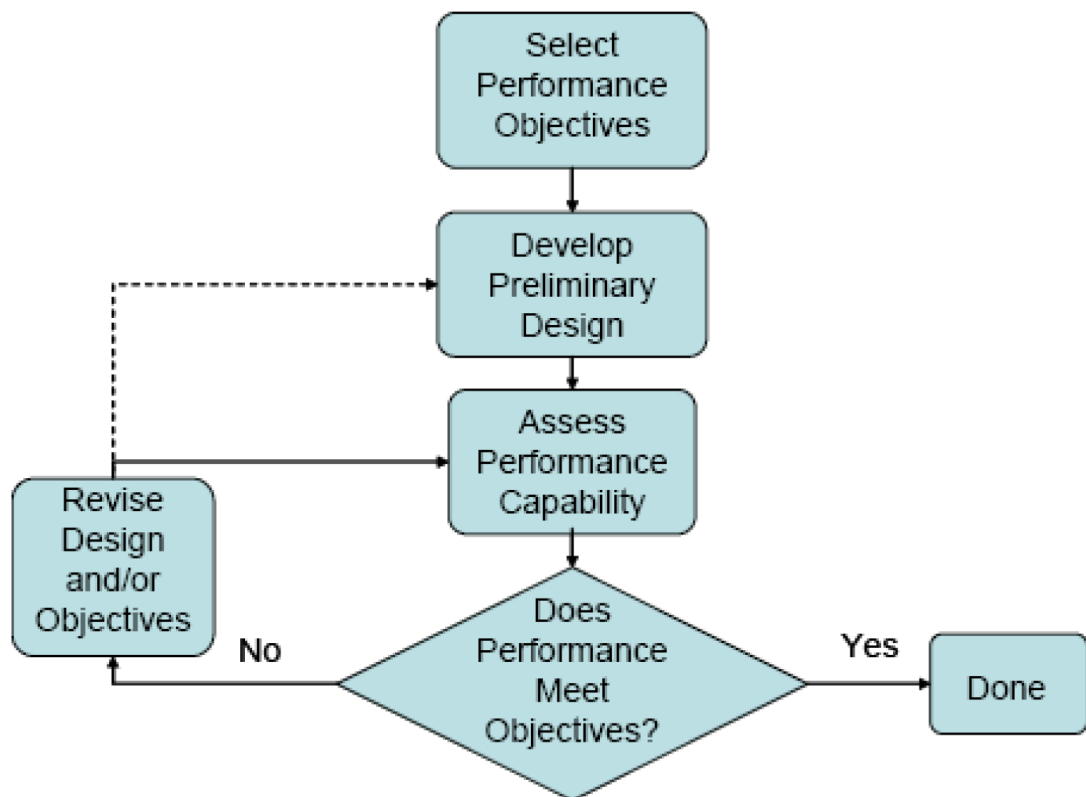


Figure 3.4 : Performance-Based Design Flowchart

3.2 Endurance Time Method

The idea of the Endurance Time method is somewhat similar to the method used by cardiologists to evaluate the condition of the heart, known as a stress test. In stress tests, the patient is asked to walk on the treadmill with a variable slope and speed. The test starts with a low slope and low speed condition. During the test, the slope and speed are increased gradually, while the physical and biological condition of the patient, such as blood pressure, heart beat rate, etc. are monitored. The test is continued until signs of distress or abnormal conditions are observed. The heart condition is then judged based on the speed and slope level that could be tolerated.

In the Endurance Time method, nearly the same concept is applied. The idea behind the ET method is roughly, to put the structure on a ramp-like accelerogram and see how far it can go. The structure is subjected to a standard accelerogram with intensifying dynamic demand. The specified performance indices are monitored until they reach a predefined maximum value. The performance of the structure is judged based on the time when the damage limit index is exceeded.

3.2.1 Endurance Time Concept

The concept of Endurance Time method (ET) can be best explained by considering a hypothetical experiment. Assume that three model buildings with unknown seismic resistance characteristics are to be investigated with reference to their performance against collapse in high intensity earthquakes. Now consider that these models are put on the shaking table and are fixed to it. The experiment starts by subjecting the buildings to random vibration with gradually increasing intensity. In the beginning (e.g. at $t = 5$ sec) the amplitude of shaking is quite low so all three buildings vibrate but remain stable, as shown in Figure 3.5.a. As the amplitude of vibration is increased (say. at $t = 10$ sec), a point is reached when one of the buildings collapses. It is assumed the model number one, as in Figure 3.5.b. As the vibration amplitude is further increased (say, at $t = 20$ sec) and through the time, the second structure fails. Assume this is building number 2, as in Figure 3.5.c. Further, consider that building number 3 happens to be the last building to fail in this hypothetical experiment. [15]

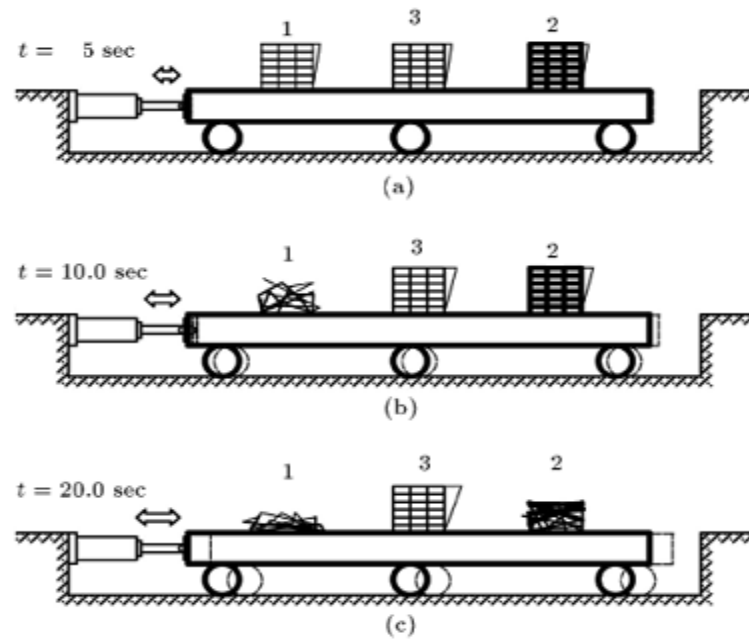


Figure 3.5 : Hypothetical Shaking Table Experiment

It is obvious that the drift range of a frame 1 is more than the other frame ranges and it is less in frame 3. Frame 1 would reach to its allowable values in second five and Frame 2 in second 10 and the Frame 3 in second 20. The allowable value is obtained 0.005 by considering $R=6$. [5]. (R is the Structure Response Modification Factor)

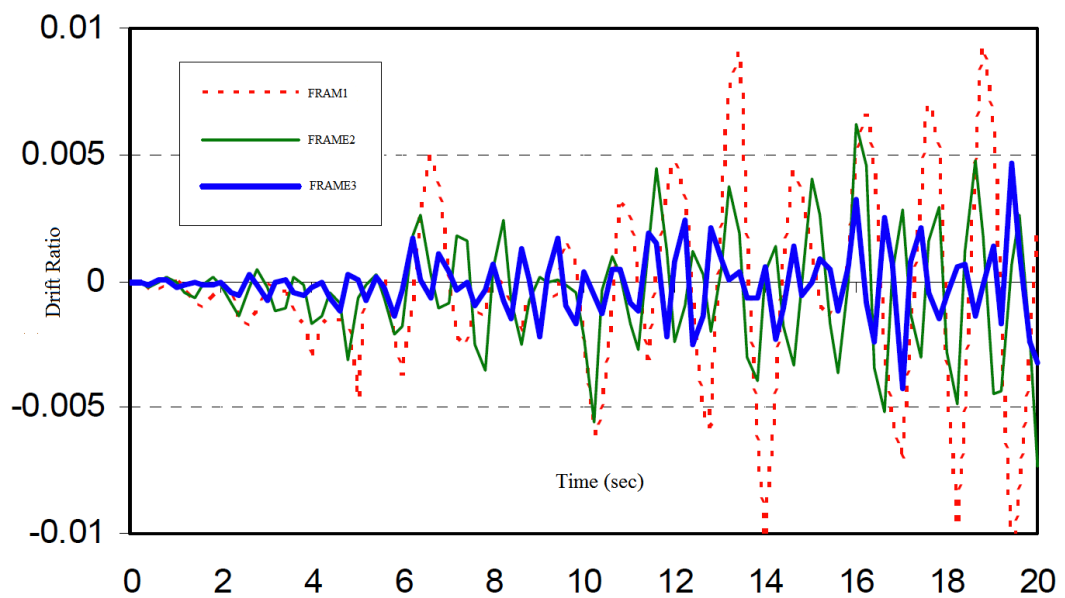


Figure 3.6 : Drift Ratio of Experienced Frames under Intensifying Shaking

In order to analyze and design the structures, the maximum drift ratio values are usually important. For example, drift ratio value for Frame 2 is 0.00237 in second 12. If the absolute values of drift considered as a design criterion, the maximum values up to second 12 is equal to 0.0056. Therefore, the drawn diagram would be applicable in the structural analysis and designing.

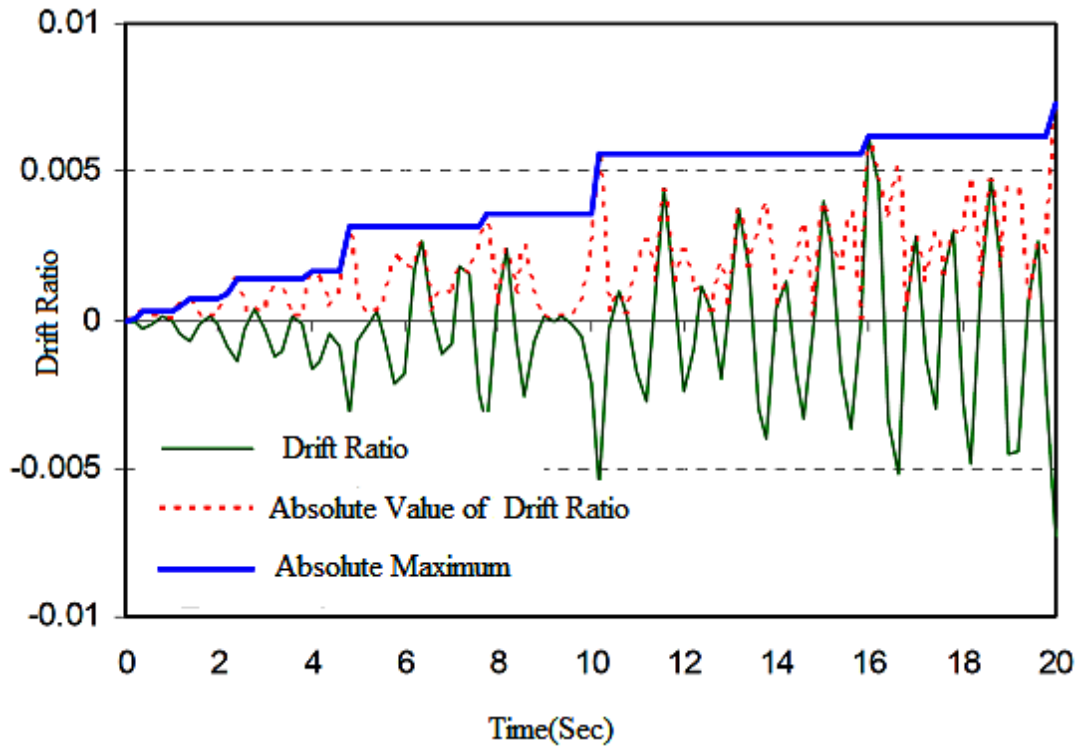


Figure 3.7 : Maximum Drift Ratio Diagram for Frame 2

The diagrams of frames 1&3 are in Figure 3.8. It is clear that, the shaking range is less at the beginning and all three frames are stable. After a short time, Frame 1 fails as the amplitude of the vibration increased. At the $t=10\text{sec}$, the Frame 2 will fail and finally Frame 3 fails at $t=20\text{sec}$. Frames that can endure the imposed intensifying acceleration function for longer are expected to be capable of sustaining stronger seismic excitation.

Considering the times at which the models have been failed and regarding to the lateral loads induced by the shaking table somehow correspond with earthquake loads, one can run the three frames according to their seismic resistance. Hence, here, the endurance time of each frame against intensifying shaking can be considered as the seismic resistance criterion [5].

In this method, the structures are subjected to the series of intensifying accelerograms in such a way that structures fail. The resistance criterion (e.g. acceleration response, member internal stress, and displacement) diagram is drawn through time. Figure 3.8 [5]

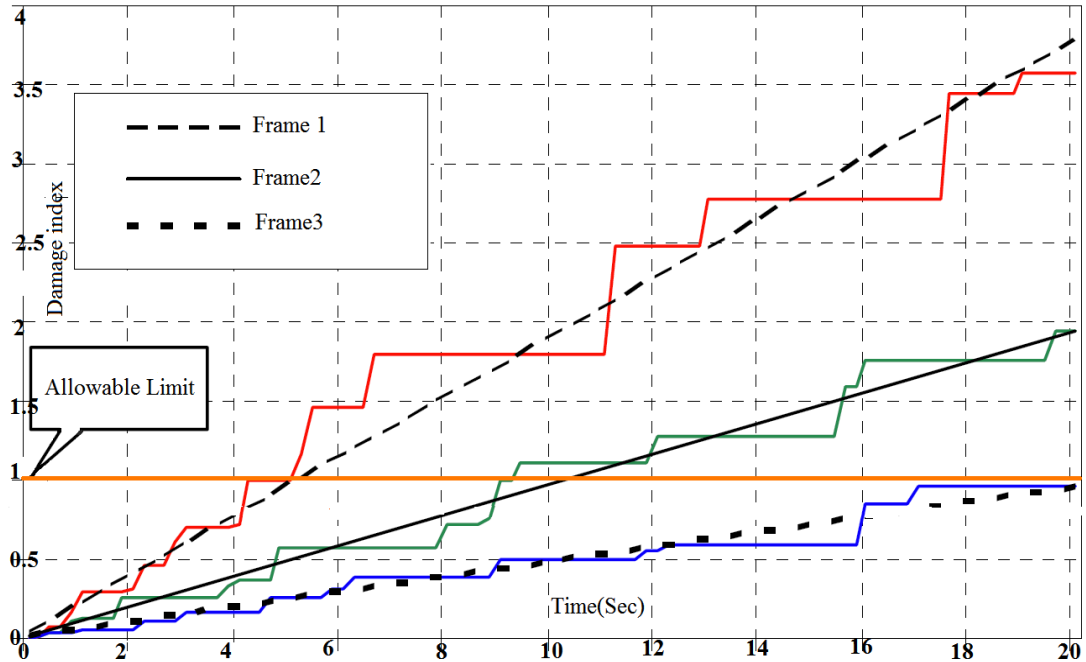


Figure 3.8 : Structure Response Diagram under Acceleration Function

It is required a method to depict the above idea in order to obtain numerical values. Hypothetical experiment of the real structures under shaking table is an old method, which has two serious problems. First, this method takes too long time and costly. Second, it is not applicable in the complex and huge structures. Therefore, a theoretical method with series of acceleration functions shall be used for analyzing due to achieve the purpose of the Endurance Time method. [5]

Now, it is asked which structures have the minimum conditions based on the ET method and according to the resistance and the serviceability criterions. Also, which structure is the standard one or which one is the stronger or the weaker?

In order to answer these questions, various parameters are considered as following:

- Allowable limits of different codes based on the serviceability and resistance are considered as an Endurance Time method limitation and range.
- The minimum required time for the structures stability based on the considered criteria is equal to 10 seconds. Therefore, second 10th is supposed as a target time in the Endurance Time Method.

According to the two above criteria, if it is supposed that the maximum Damage Index limit is equal to 1.0, the endurance time of the frame 1 is in the 5.2th second, according to the Figure 3.8. Therefore, the Frame 1 will fail before the target time. As a result, this frame cannot satisfy the design criteria and is weak but the Frame 2 will fail approximately in the target time, satisfy the required minimum endurance, and is standard. On the other hand, Frame 3 will fail in the time more than the required one, thus the endurance of the Frame 3 is more than the target time and is stronger than standard frame. [5]

Besides ETAFs, considering an appropriate target time is important in the evaluation of structures by the ET method. Target time is defined as the time when the effective response spectrum of the induced dynamic excitation matches the required design spectrum at which performance requirements are to be met. In linear analysis, the selection of a target time is rather arbitrary. The selection of a suitable target time should be based on the analysis purpose. In nonlinear analysis, if the target time is set too low, the induced excitation will increase too quickly, and the structure will not experience a reasonable number of cycles matching the real situation. On the other hand, if the target time is set too high, the number of nonlinear cycles will be much higher than it occurs due to actual ground motions (Estekanchi et al., 2007). In this study, a target value of 10 s has been utilized for the ET analysis. The optimal target time in nonlinear analysis should somehow have correlation with the duration of representative ground motions. Preliminary research shows that a target time of 10 s that was originally selected based on engineering judgment is a reasonably good selection for design-level earthquakes (Bazmooneh (2009)).

3.2.2 Accelerograms Properties

As mentioned in the previous section, the accelerations using in the ET method for the Time History Analysis shall have two main characteristics, which are dominated from other accelerations, intensifying amplitude and calibrated accelerograms that are explained briefly next.

The choice of an appropriate accelerogram functions is fundamental to the successful implementation of the ET concept. The ideal input function is the one that results in higher consistency and best correlation between the Endurance Time analysis results and the actual or well-known performance characteristics of the structures subjected to the earthquakes.

Determination of the most appropriate and optimal intensifying accelerogram functions should, in itself, be the subject of extensive research work. However, in this research, a function based on engineering judgment and some elementary calculations will be proposed so that the concept and application of the ET method can be explained. [15]

Three suggested intensifying accelerogram functions are shown below.

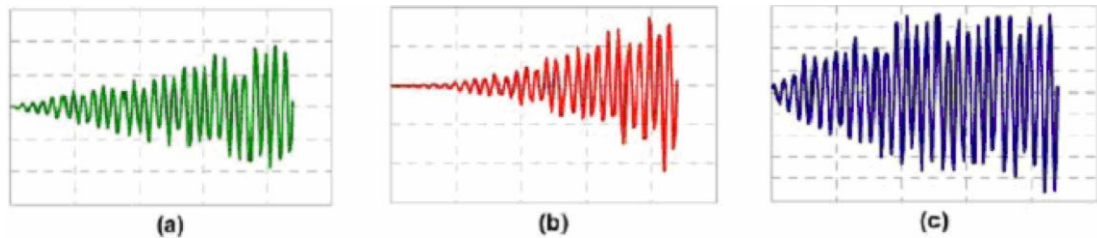


Figure 3.9 : Suggested Intensifying Accelerograms

As we know, the acceleration records in a nature earthquake would have three main characteristics; amplitude, endurance, and frequency content, which depend on the seismic intensity, seismicity of the site and soil conditions. [16] In order to produce accelerograms similar to the nature records, the mentioned three parameters must be defined properly. The accelerograms amplitude are discussed and elucidated they are intensifying functions. The seismic endurance is not considered because the amplitude intensification of the accelerogram functions would continue up to the structure failure. The third parameter is the frequency content of the accelerogram functions, which is obtained by the random pattern method. This method is explicated in the next section.

3.2.3 Producing Intensifying Accelerogram Functions

Accelerogram functions are obtained by using the non-stationary Random Pattern Method in the range of the amplitude and the frequency content. In this method, the The vibration function may be expressed as the sum of an infinite number of sinusoidal waves as shown in the equation below. [17]

$$a(t) = \sum_{i=1}^N A_i \cdot \sin(w_i t + \varphi_i) \quad (3.1)$$

where:

A_i : i^{th} Amplitude

φ_i : Phase angle of i^{th} sinuous component

w_i : i^{th} angle frequency

If the amplitudes and the phase angles relating to the sinuous components were considered as an amplitude vector and the phase angle vector, by making the different phase angle vector with constant amplitude vector, the various motions with the similar frequency contents would be produced. The output vectors are Stationary and Ergodic.

The first generation of accelerograms to be used as a dynamic input in the ET method is produced using random numbers with a Gaussian distribution of zero mean and a variance of unity [15]. A stationary random accelerogram is generated using time distance $\delta t = 0.01\text{sec}$ and selected points $n = 2^{11} = 2048$ with $\text{PGA} = 1$ shown in Figure 3.10. Duration of the accelerogram is equal to $\delta t \times n = 20.48\text{sec}$.

The produced functions by the mentioned method are stationary while the nature seismic accelerations are not. Therefore, the stationary functions shall be modified to the non-stationary mode by multiplying by the proper spectrum function. This spectrum function should be compatible with earthquake intense and the seismicity of the site, such a spectrum function is depicted below, and relative equation is suggested. [5]

$$z(t) = f(t).a(t) \quad (3.2)$$

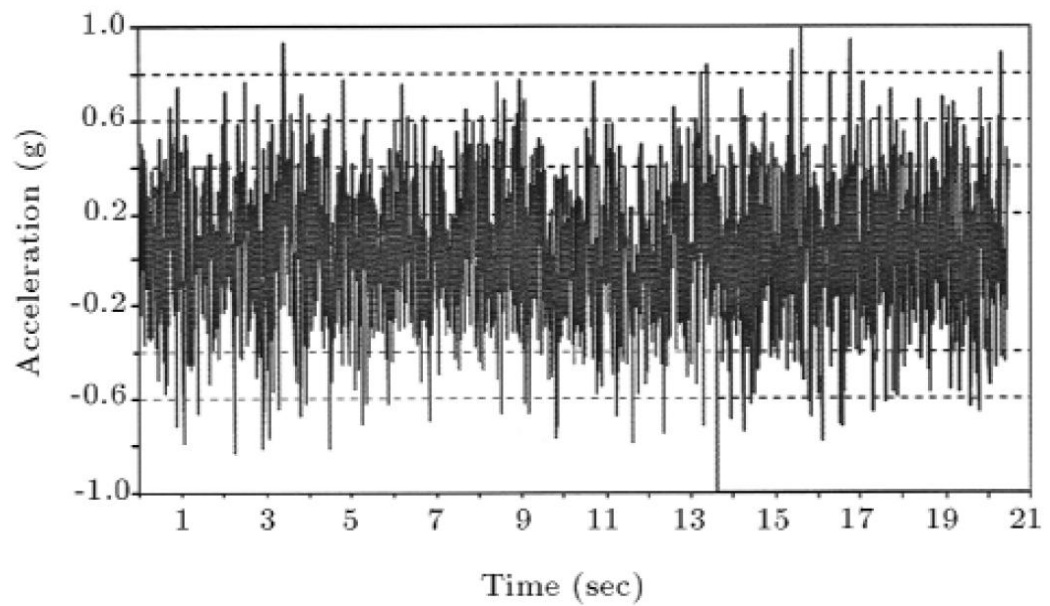


Figure 3.10 : A typical Random Acceleration with PGA=1

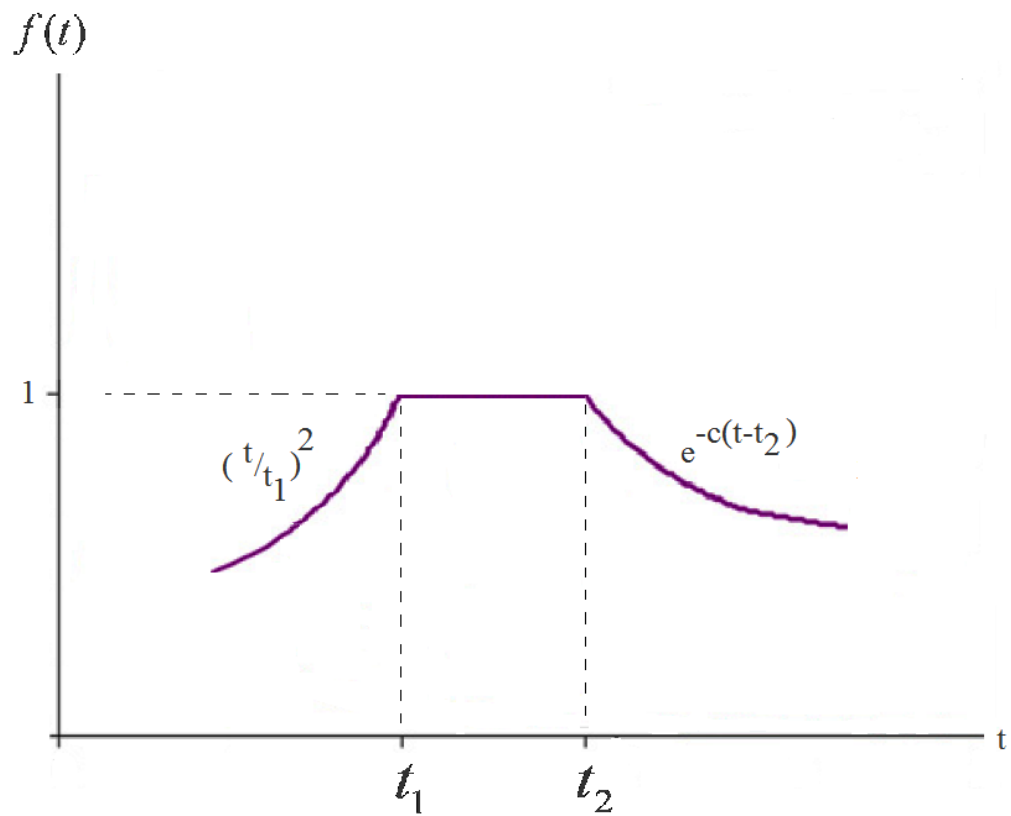


Figure 3.11 : $f(t)$ Spectrum Function

The frequency content of the random accelerogram $a(t)$ that is statistically similar to a white noise is then modified, in order to resemble actual earthquake accelerograms. For this purpose, the filter functions given by the following equations are applied to the Fourier transform of $Z(t)$ function. The application of these filter functions is explained by Clough and Penzien [17].

$$Z(i\omega) = F(z(t)) \quad (3.3)$$

$$A(i\omega) = Z(i\omega).H_1(i\omega).H_2(i\omega) \quad (3.4)$$

$$H_1(i\bar{\omega}) = \frac{1 + 2i\xi_1 \frac{\bar{\omega}}{w_1}}{(1 - \frac{\bar{\omega}^2}{w_1^2}) + 2i\xi_1 \frac{\bar{\omega}}{w_1}} \quad (3.5)$$

$$H_2(i\bar{\omega}) = \frac{(\frac{\bar{\omega}}{w_2})^2}{(1 - \frac{\bar{\omega}^2}{w_2^2}) + 2i\xi_2 \frac{\bar{\omega}}{w_2}} \quad (3.6)$$

where;

F is the Fourier transform and $A(i\omega)$ is the acceleration in the frequency domain. The frequencies higher than 100Hz are filtered out by considering:

$$\xi_2 = 0.2, w_2 = \frac{2\pi}{0.1}, \xi_1 = 0.2, w_1 = \frac{2\pi}{0.5}$$

Then, the $A(i\omega)$ function will be transferred into the time domain by the inverse Fourier transform and the $a(t)$ acceleration function is obtained. The resulting response spectra is compared with the code response acceleration, if the response spectra is compatible with typical seismic code response acceleration, that response spectra function is accepted as a calibrated accelerogram function, if not, the Fourier transform must be modified by below equation. Note that any given design response spectrum can be used for this purpose.

$$A^{new}(w) = A^{old}(w) \cdot \frac{S_a^T(w)}{S_a(w)} \quad (3.7)$$

where:

$A^{old}(w)$ is the Fourier transform of the acceleration function, $S_a^T(w)$ is the target spectrum and $S_a(w)$ is the acceleration response spectrum which are calculated before, at the end the inverse transform of $A^{new}(w)$ is computed for the acceleration function in the time range. Hence, a new accelerogram function with the higher compatibility is produced. [15]

A sample frequency content resulted from the accelerogram shown in Figure 3.10 is depicted Figure 3.12

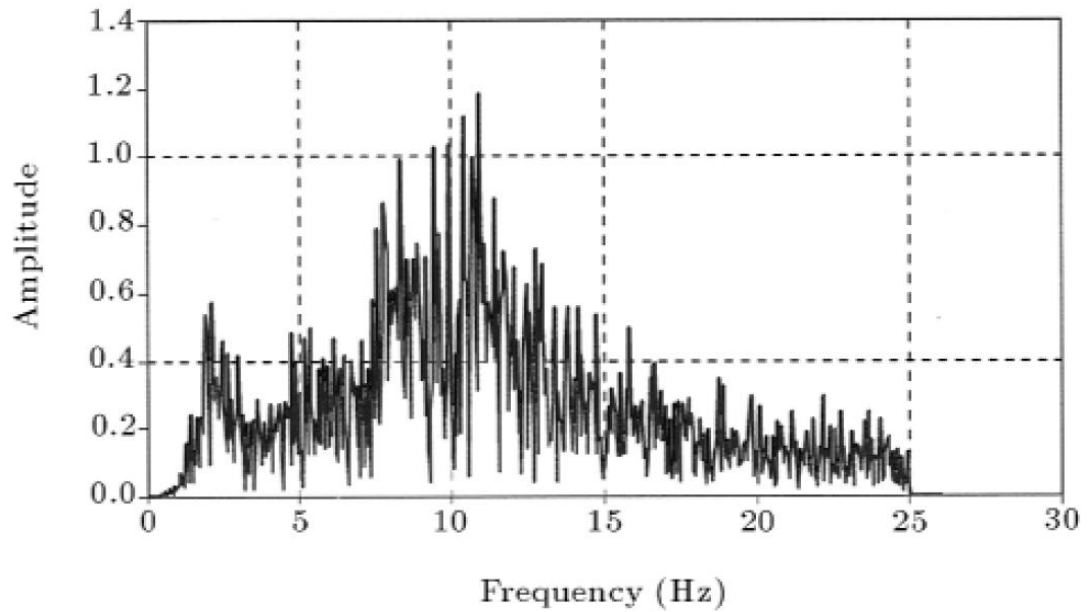


Figure 3.12 : Filtered Frequency Content

In this thesis, the response spectra of the Iranian National Building Code (Standard 2800) (the same as API and FEMA) have been used as a sample. The resulting frequency content of a response spectra compatible accelerogram, after several cycles of stepwise modification, is shown in Figure 3.13

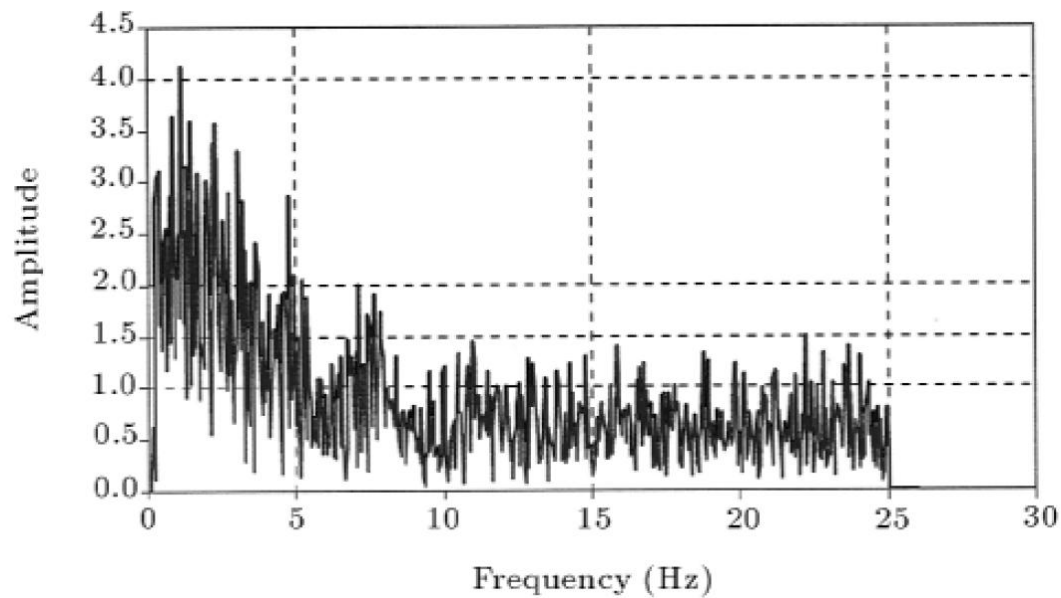


Figure 3.13 : Modified Frequency Content

The Target Time that is the design, checking, and comparison criterion is the important parameter in the calibration procedure. This time assumed 10 second in the previous sections in other word; the minimum structure stability criterion must be 10 second. The base shear in the static and spectrum analysis is obtained base on the standard spectrum somehow. The structure acceptance and rejection criterion in the static analysis is based on the correlation between this base shear and Damage Indices, and is based on the standard spectrum in the spectrum analysis. It is expected that the structure results from static analysis and ET analysis in the Time Target be equal to each other, if the accelerogram functions response spectrum in the ET method is equal to the standard spectra in the Target Time.

Acceleration response, which is compared with the code response spectrum curve, is depicted in Figure 3.14. The convergence is assumed well enough for the purpose of explanation of the ET concept. Further calibration of the accelerogram is investigated by Estekanchi and colleagues. [15]

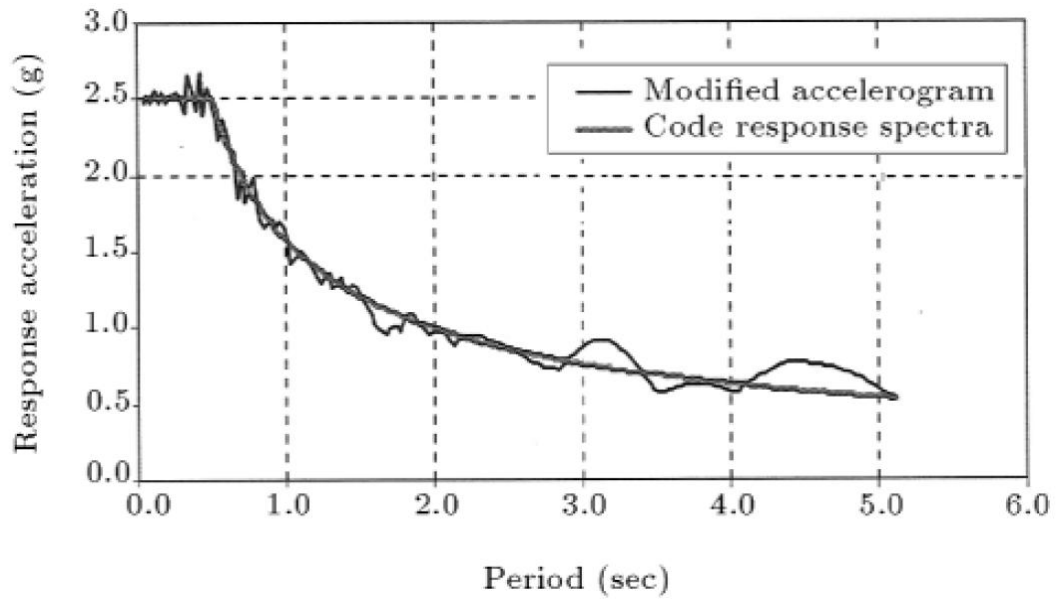


Figure 3.14 : Response of Modified Accelerogram Compared to Codified values

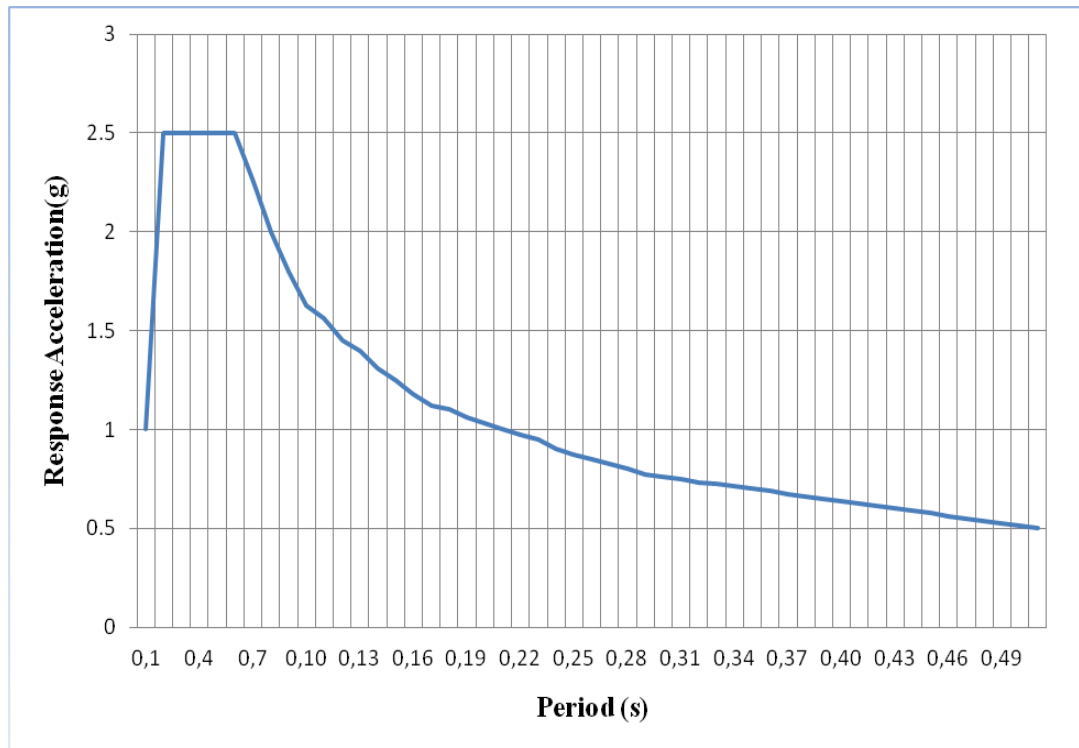


Figure 3.15 : API & INBC Code Response

In the next step, the acceleration values are adjusted for Target values of velocity and acceleration, which, in this case, are set to zero. Finally, the acceleration values are multiplied by a profile function, which, in this sample, is a linear one starting from zero and reaching a value of 1.00 at $t = 10$ sec. Figure 3.15

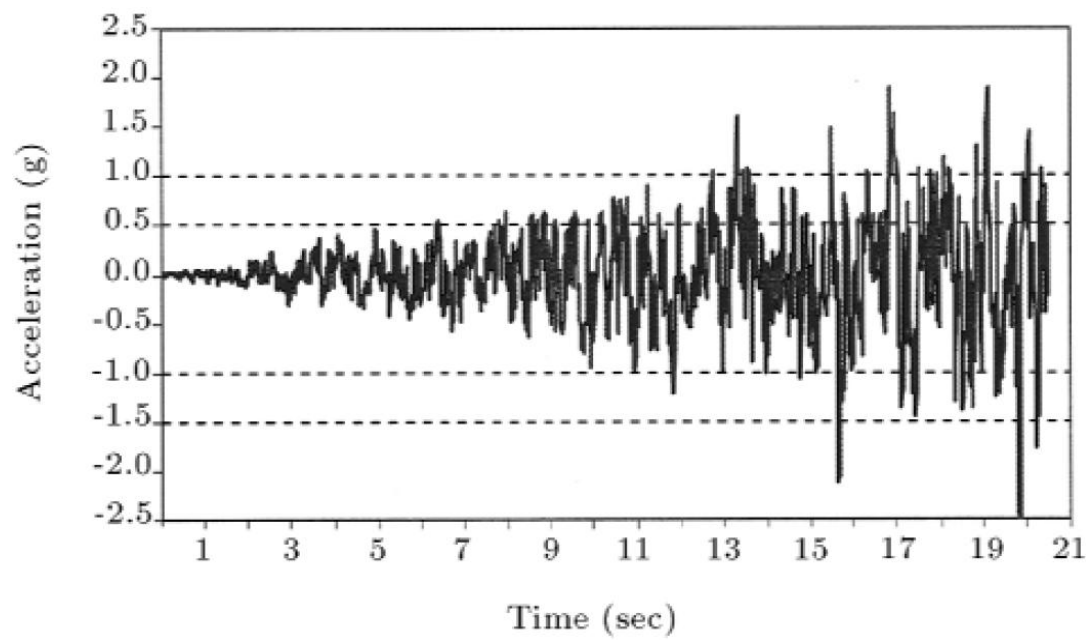


Figure 3.16 : Final Increased Accelerogram

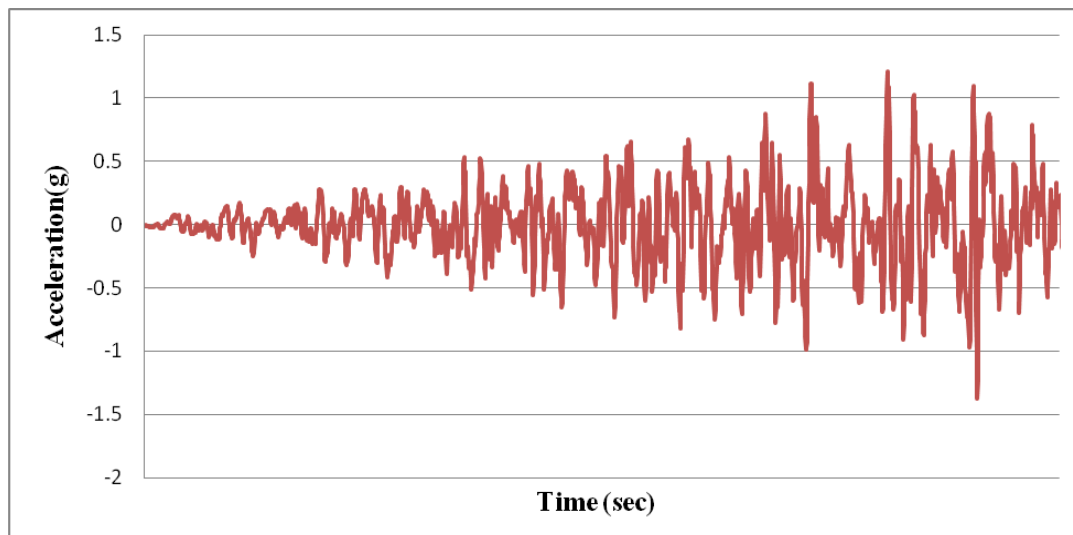


Figure 3.17: ETA20f02 Acceleration Function

Since, ET acceleration functions are supposed to be intensifying with time and in order to calibrate the accelerogram functions in 10th second, the resulted response spectra must be compatible to the coded spectra, hence the below equations are suggested.

$$S_{aT}(T, t) = \frac{t}{t_{Target}} S_{aC}(T) \quad (3.8)$$

$$S_{uT}(T, t) = \frac{t}{t_{Target}} S_{aC}(T) \times \frac{T^2}{4\pi^2} \quad (3.9)$$

where:

"T" is the period of free vibration, "t" is a time, $S_{uT}(T, t)$ is a target displacement response, $S_{aT}(T, t)$ is a target acceleration response, $S_{aC}(T, t)$ is the codified design acceleration spectrum. Problem of generating an accelerogram with such characteristics was approached by formulating it as an unconstrained optimization problem in the time domain as follows: [5]

$$Min F(a_g) = \int_0^{T_{max}} \int_0^{t_{max}} \left\{ [S_a(T, t) - S_{aT}(T, t)]^2 + \alpha [S_u(T, t) - S_{uT}(T, t)]^2 \right\} dt dT \quad (3.10)$$

where:

a_g is the ET accelerogram being sought and α is an optimization weight parameter, set to 1.0 in this study. According to the mentioned equations, some records have been optimized to fit average RS of seven records for soil C FEMA 440 selected by Estekanchi and colleagues. Optimization is for 200 Ts in 0-5 sec plus 20 points from T=6 to T=50 sec. [5]

3.2.4 Accelerogram Selection

In this thesis, the purpose of the Time History analysis method is the evaluation of the structures performance level rather than designing the structures such as jacket or etc. Therefore, the earthquakes spectrums and the average of them shall be the best evaluation of the coded spectrum. Since, the non-linear analysis depends on the seismic intensity and magnitude; it is recommended that seismic spectra be close to the coded spectra, too.

One of the main goals of this study is to validate the ET method by comparing its results with the nonlinear response of the structure subjected to the ground motions. Therefore, the three ET acceleration functions used in this thesis (ETA20f set; Fig. 3.16) are generated in such a way that their response spectra remain proportional to that of the average of seven ground motions recorded on a soil type C FEMA440 condition (same as type C in API 2000 and type II in INBC) [18]. These ground motions are selected from 20-earthquake ground motions used in the ATC-55 project and are scaled to be more compatible with seismic code response spectra [18,19]. The average response spectrum is smoothed and is used for generation of the ETA20f set Figure 3.17. The specifications of the ground motions of this set are shown in Table 3.4. The response spectrum of any window of the ETA20f set of acceleration functions from $t_0 = 0$ to $t_1 = t$ resemble that of the average response spectrum of the seven ground motions (GM) with a scale factor that is proportional with time (t) [6]. This scale factor is equal to 1.0 for $t_{Target} = 10$ s in this study, according to FEMA 440 recommendation.

Table 3.4 : Description of the Ground Motion Set Used in This Study

Date	Earthquake name	Magnitude (Ms)	Station number	Component (deg)	PGA	Abbreviation
28/06/92	Landers	7.5	12149	0	167.8	LADSP000
17/10/89	Loma Prieta	7.1	58065	0	494.5	LPSTG000
17/10/89	Loma Prieta	7.1	47006	67	349.1	LPGIL067
17/10/89	Loma Prieta	7.1	58135	360	433.1	LPLOB000
17/10/89	Loma Prieta	7.1	1652	270	239.4	LPAND270
24/04/84	Morgan Hill	6.1	57383	90	280.4	MHG06090
17/01/94	Northridge	6.8	24278	360	504.2	NRORR360

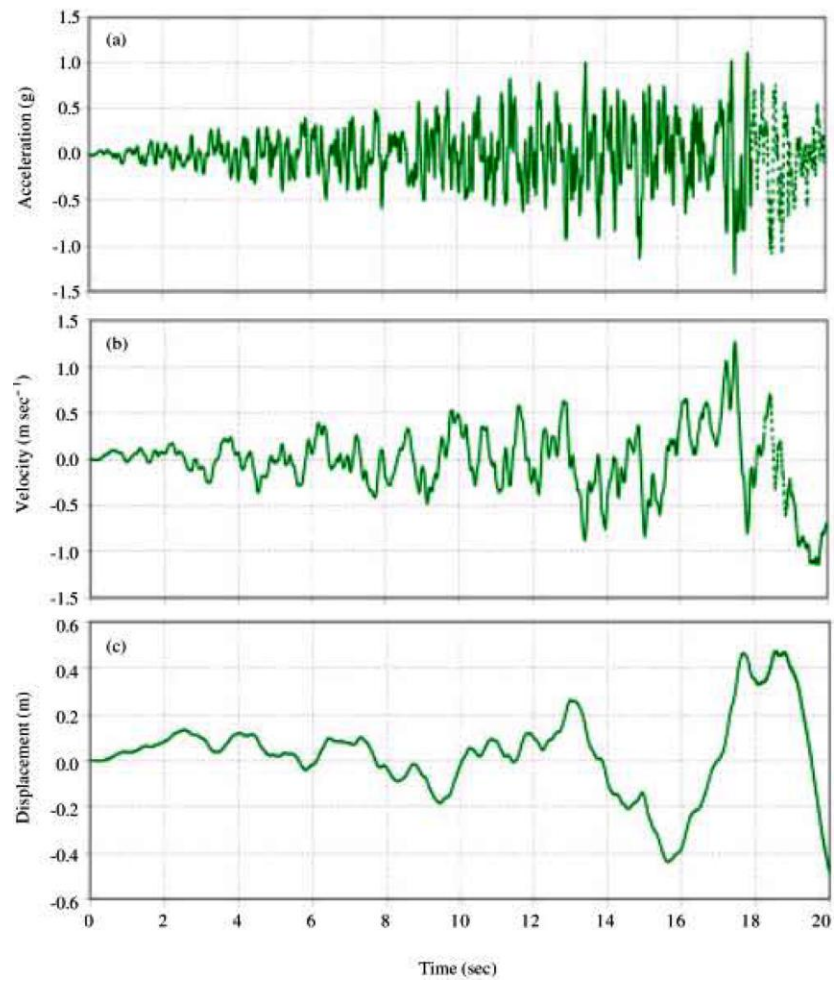


Figure 3.18 : (a): ETA20f01 Acceleration Function, (b): ETA20f01 Velocity and (c): ETA20f01 Displacement Function [6]

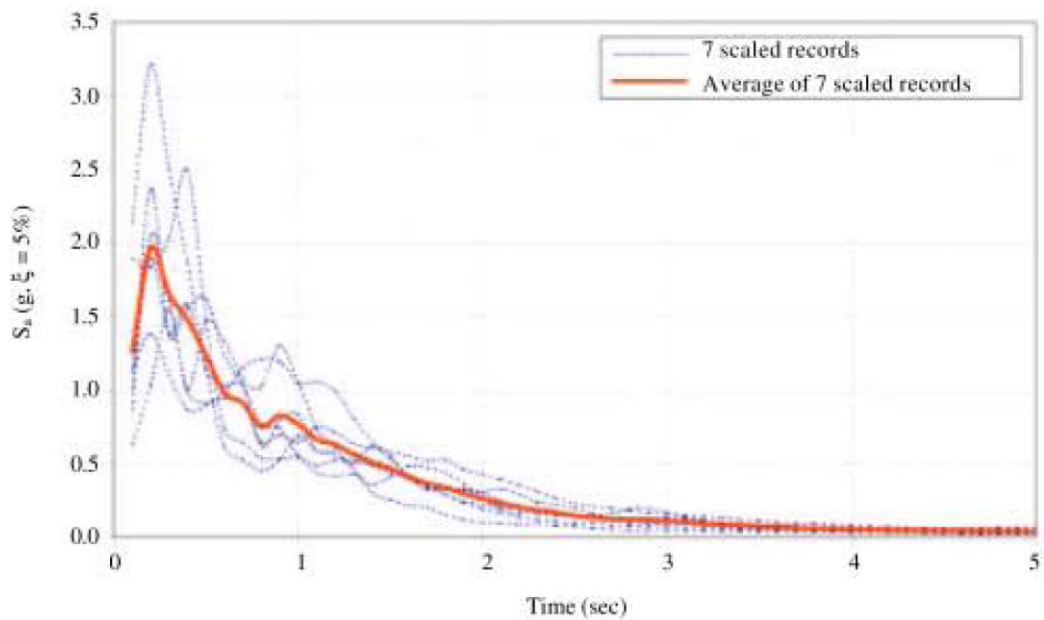


Figure 3.19 : Total Acceleration Response Spectra of 7 Scaled Accelerograms and their Average Spectrum [6]

3.3 Structural Analysis

The SPD25 jacket is analyzed by three different methods and the results are compared to each other. The two well-known methods, Time History analysis, and Pushover analysis are not explained and just the results are depicted at the end but the new Endurance Time method by Estekanchi is explained and the results are evaluated by two other methods finally. The necessary calculations are shown in Appendix C for all methods.

3.3.1 Static Pushover Analysis

Pushover analysis has recently gained considerable popularity in the seismic assessment of structures. An appropriate engineering judgment can be obtained from the global pushover curve.

3.3.2 Time History Analysis

Time history analysis is executed usually by the modal or direct integration methods. In this study, the direct integration method is used by SAP2000 software. In this method the time value and duration must be defined. The more defined values and shorter duration, the more exact results will outcome, but the analysis procedure will be longer. In this thesis, seven nonlinear time history analysis are done by seven records in various correction factors.

3.4 Structures Performance Evaluation Based on FEMA356

According to the FEMA 356, story drift ratio shall be considered as a first index in order to define performance levels, the drift values must be defined and compared with Table 3.2. A simple comparison of FEMA 356 and FEMA 351 explicate that the results of these two codes are the same and FEMA 356 is more simple and faster to use. [5]

The other parameter in FEMA 356 as an index of performance level definition is member plastic rotation. In this thesis, the beams are supposed weak and the columns are strong, so that the plastic hinges are in beams.

$$\text{Beam : } \theta_y = \frac{ZF_{ye}L_b}{6EI_b} \quad (3.11)$$

$$\text{Column : } \theta_y = \frac{ZF_{ye}L_c}{6EI_c} \left(1 - \frac{P}{P_{ye}}\right) \quad (3.12)$$

where;

I = Moment of inertia

θ_y = Yield rotation

E = Modulus of elasticity

Z = Plastic section modulus

P_{ye} = expected axial yield force of the member

F_{ye} = Expected yield strength

L = Member Length

P = Axial force in members at the target displacement for nonlinear static analysis

The member plastic rotation would be obtained by nonlinear analysis and then the performance levels can be defined according to FEMA 356.

The total structure performance level can be obtained by considering the structure performance levels resulted by story drift ratio and member plastic rotation. The worst results must be chosen as an answer. According to the structural performance level, the below items would be discussed to define the structure's PL in FEMA:

- The mentioned PLs in FEMA and other codes are separate levels, which has no certain distance between them, on the other word when the structure is in IO level and the other structure is in LS level, there is no judgment about these structures real performance.
- These performance levels have no numerical concepts. It means that if two structures are in on PL, the comparison of these two structures isn't possible. This subject is considered when one structure will be evaluated under various accelerations e.g. if the structure responses are compared under NRORR and LPGLOB000 accelerations, it is resulted based on the FEMA PLs that this structure is in LS performance level in the both seismic and no more comparison is possible.
- The main problem of this method is that, all mentioned calculation procedure can be done only for the certain acceleration and finally the structure performance is obtained just under this acceleration. In addition, no more judgment is possible about structure under other accelerations.

Therefore, in order to evaluate the performance of structures, the performance of all elements should be checked by observing their plastic deformations and comparing the values with the acceptance criteria. Since different limits are set on these parameters for various elements in each PL, it is difficult to compare the performance of different elements and clearly define the critical one. Thus to accurately evaluate the performance of a structure, one should create a target curve for each mentioned index and element, and compare the related response curve with that target curve. In this way, even though the performance can be identified, the specification of the critical index is not a simple matter. [5]

3.5 Performance-Curve

As previously mentioned, the reference curve is a concept by means of which the specific properties of the ET method can be readily put into use in the performance based design procedure. Using the reference curve is an appropriate way to evaluate the performance of structures in the ET method. In fact, the reference curve will be used as the criteria curve for the ET response curve and the performance of a structure at different excitation levels can be evaluated by comparing the ET response curve with the target curve. Since, the target and the ET response curves are compared in a single chart, the horizontal and the vertical axes of the target curve should be defined in the target response curve (or simply target curve). These axes shall be matched with the corresponded axes in the ET response curve, i.e. the time in seconds is on the horizontal axis, and a drift index is on the vertical axis. On the other hand, the reference curve should be inclusive of a relation between the performance levels and the corresponding performance criteria. Thus, the challenge is to correlate the performance levels to the endurance times. In other words, the first step in the process of creating the reference curve is to identify the respective endurance time of each PL. To do so, the procedure shown in Figure 3.18 should be followed systematically. [5]

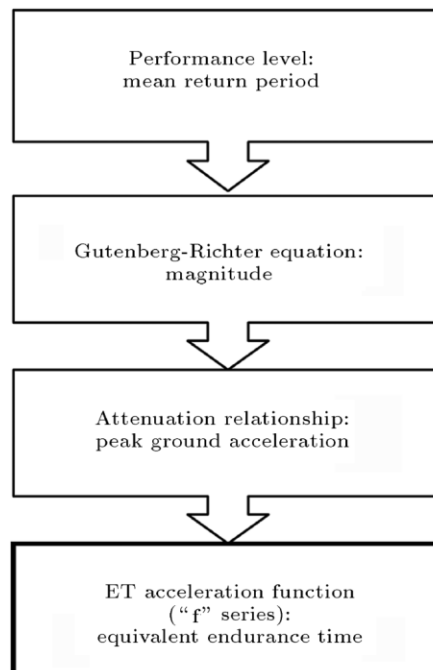


Figure 3.20 : Reference curve Procedure

Since the main purpose of this study is to illustrate the basic concept of ET on the jacket, the following approximate procedure can be considered as appropriate. The first stage is to define the mean return period which will vary based on the structures types and our expectation from structure performance for example in an intensive earthquake it is expected that the performance level of a significant structure such as hospital be IO or LS. So, in this thesis the jacket return period is considered as FEMA recommendation for important structure. It is mentioned in Table 3.1 in previous sections. Now, the relation between these return periods and endurance time of each performance level will be obtained by the following diagram.

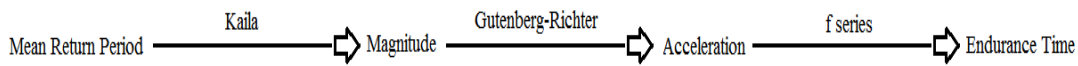


Figure 3.21 : Endurance Time Evaluation Diagram Using Return Period

The corresponding magnitude of each earthquake hazard has been obtained, first, using the Gutenberg-Richter relation as follows [5]:

$$L \log N = a - bM \quad (3.13)$$

where:

N is the return period of earthquake (years)

M is the magnitude of earthquake in Richter

b is called the “b-value” and is typically in the range of 0.8-1.2

a is called the “productivity”

' a ' and ' b ' are some parameters that severely depend on the properties of the region in which the earthquake had occurred. For Iran, the following form is recommended by Kaila [20]:

$$\log N = 6.02 - 1.18M \quad (3.14)$$

Table 3.5 : a and b Values for World Countries

b	a	Specification				Zone
1.22	6.86					Japan
1.09	5.05	115W	142E	65N	47N	East Canada
1.14	5.94	105W	135W	47N	25N	East USA
1.38	5.79	51W	105W	47N	25N	West USA
1.45	7.36	85W	120W	25N	10N	Center USA
1.11	5.60	60W	85W	6N	18S	Colombia-Peru
0.88	4.78	60W	78W	37S	63S	North Chile
0.92	4.46	60W	78W	37S	63S	South Chile
1.18	6.02	65W	48E	42N	15N	Turkey-Iran
1.10	5.45	48E	20W	50N	30N	Middle East

After that, the Peak Ground Acceleration (PGA) must be acquired from magnitude. The following formula is suggested by Gutenberg-Richter [5]:

$$\log a = -2.1 + 0.81M - 0.027M^2 \quad (3.15)$$

where:

a is the acceleration ($\frac{cm}{sec^2}$)

M is the magnitude of earthquake in Richter

This acceleration can be easily related to the endurance time. For this purpose, it is needed to specify which series of accelerograms are to be used. According to its conformability with the FEMA 440 (API and INBC), the 'f' series of accelerograms (i.e. ETA20f01-03) has been chosen. Considering this series, the equivalent endurance times in ET records, corresponding to the three mentioned PGA, can be identified. To do so, it is enough to trace the times in the ET acceleration function at which the PGAs exceed the values of the PGAs corresponding to each PL. The results of the above procedure are shown in Table 3.6.

Table 3.6 : Endurance Time Related to each PL

Performance Level	Mean Return Period	Magnitude (Richter)	PGA (g)	Endurance Time (sec)
IO	75	6.97	0.18 g	5.14
LS	475	7.78	0.38 g	10.21
CP	2475	8.495	0.69 g	19.11

It is noticeable that this procedure is not an exact one. To have an exact calculation, it is needed to obtain the hazard curve for a special site and the PGA should be acquired according to that curve.

Table 3.7, in this table, the story drift ratio for each PL is indicated. These quantities are for steel moment frames based on the FEMA 356 standard. Although these values are not intended in FEMA 356 to be used as acceptance criteria for evaluating the performance of structures, and are just some quantities which qualitatively indicate the behavior of structures at each level of performance, in this study, these values will be used as an index to show the limits of each performance level. Plastic rotation in beams is the other index, which can be used to evaluate the performance of structures.

To accurately evaluate the performance, one should obtain the values of plastic deformations in all elements (including beams, columns (pile), braces etc.), and compare them to the acceptance criteria given in the FEMA-356 standard. This means that the design of a typical structure should be such that if the 'f' series of accelerograms were applied to the structure, it would be capable of meeting the IO performance level up to 5.14 sec, the LS performance level up to 10.21 sec, and the CP performance level up to 19.11 sec.

Table 3.7 : Story Drift Ratio of each PL

Performance Level	Drift (%)	Endurance Time (sec)
IO	0.7	5.14
LS	3.5	10.21
CP	5	19.11

Based on the above discussion, the performance curve has been drawn. The vertical axis should introduce the performance level index such as a story drift ratio or the plastic rotation. According to the FEMA recommendation, the drift ratio or the plastic rotation of each performance level should be compared with the endurance time of that performance level. The primary performance curve of the drift ratio is depicted in Figure 3.22. [5]

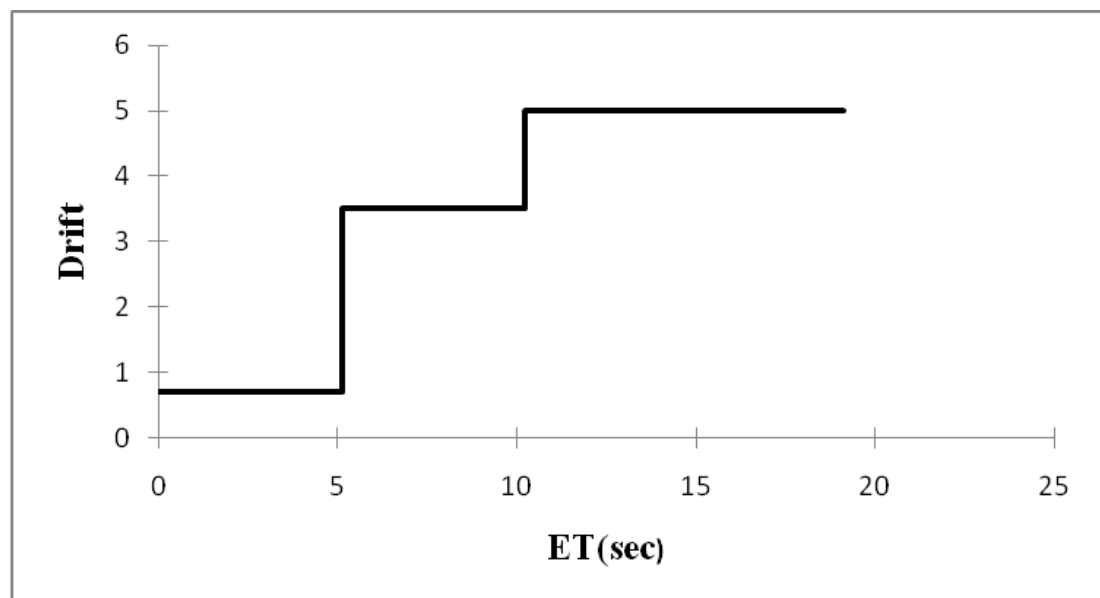


Figure 3.22 : Primary Performance Curve of Drift

The drawn curve can just solve one of the mentioned problems, the evaluation of the structure under the various accelerations, but the separation of the performance levels has been not solved yet. In order to solve this problem, it is supposed that there are several other performance levels between the FEMA recommended levels. Based on this assumption, the general modification will be occurred in Figure 3.22, the distance between the levels would be supposed contiguous. Hence, the different methods are suggested by the standards such as curved levels or stair stepping levels. Since, the ET accelerograms intensify linearly, the best mode would be linear one, on the other word, the linear performance curve means that the more weak earthquake, the better structural performance will be expected. Therefore, the performance curve will be modified as Figure 3.23. [5]

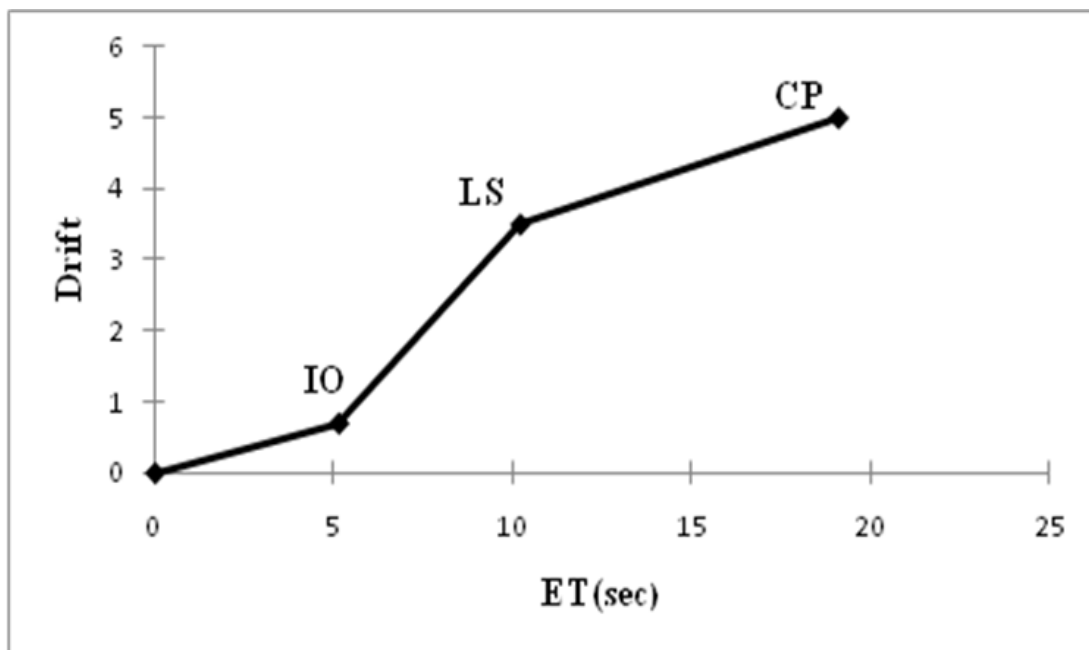


Figure 3.23 : Modified Performance Curve

The maximum story drift ratio can be used instead of story drift in the Figure 3.23. If the story drift ratio is considered 0.7% as a maximum value (IO Level), the performance curve will be modified as following. [12]

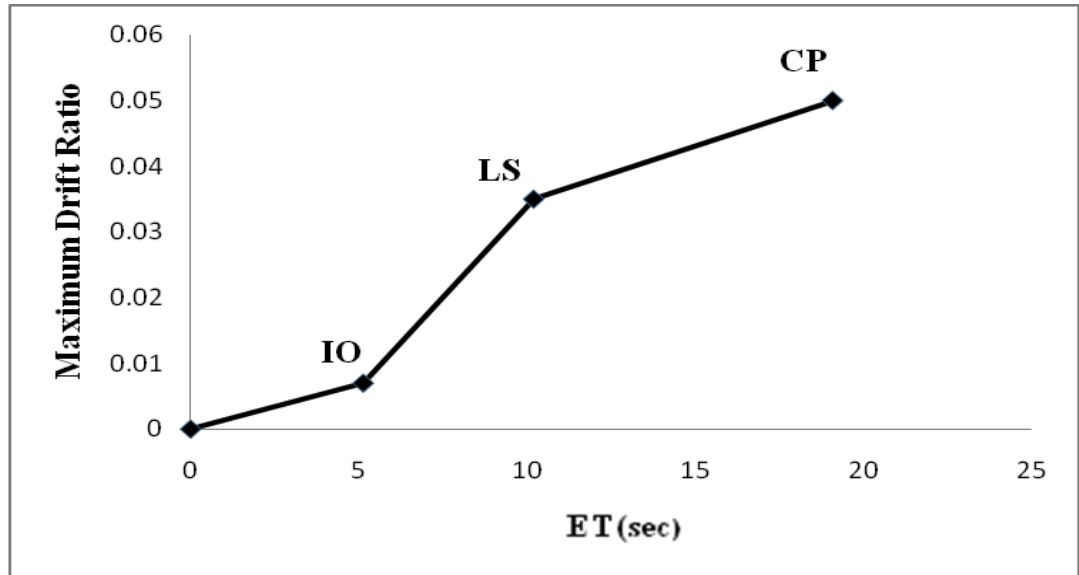


Figure 3.24 : Maximum Drift Ratio Performance Curve

Such a performance curve can be drawn for other indices such as the member plastic rotation but the FEMA consideration is three modes, which depend to the member lateral buckling strength explicating in Table 3.8. In this thesis, it is assumed that the hinge plastic will occurred in the beams first. So the relevant equations are mentioned here and the rest are referenced to the FEMA 356 code.

Table 3.8 : Beams Acceptance Criteria for Nonlinear Procedure in FEMA

		Performance Level		
		IO	LS	CP
a	$\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	$1\theta_y$	$6\theta_y$	$8\theta_y$
b	$\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	$0.25\theta_y$	$2\theta_y$	$3\theta_y$
c	Other	Linear Interpolation		

There is a main problem in order to draw the performance curve for the plastic rotation, one should sketch three separate curves for each modes or somehow make a relation between these three modes to produce the unit curve. According to the Mr. Mirzaee's true and false operation, the below results are come out.

For the “a” mode, the best simulation of three performance curves will be occurred when the rotation ratio to the maximum rotation causing the failure in the member is $11\theta_y$, and for the “b” mode, it will be the best when the rotation ratio to the rotation causing the reduction in the member strength is $4\theta_y$ [5]. The Figure 3.25 is the depicted performance curve for the three modes rotations.

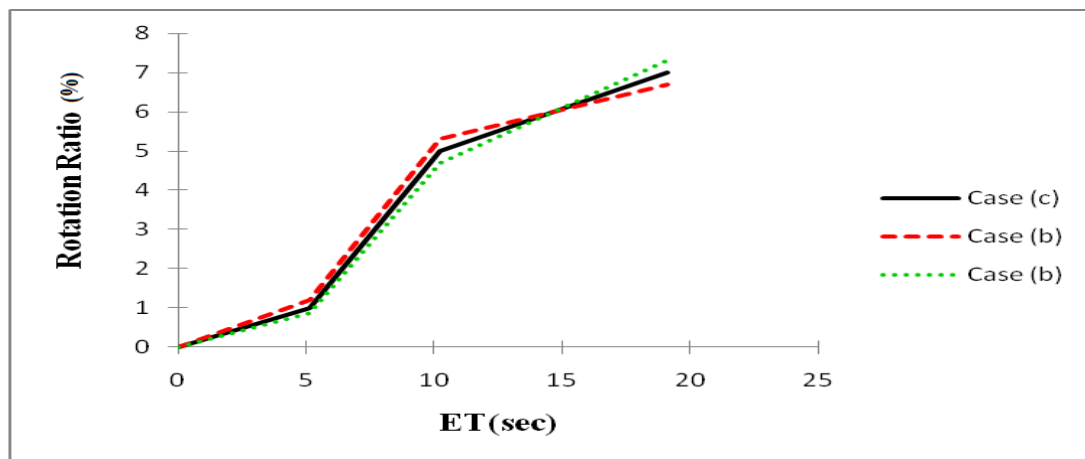


Figure 3.25 : Performance Curves for Three Plastic Rotation Modes

Since, these curves are close by each other, the average of them can be considered as a relevant performance curve of the plastic rotation index.

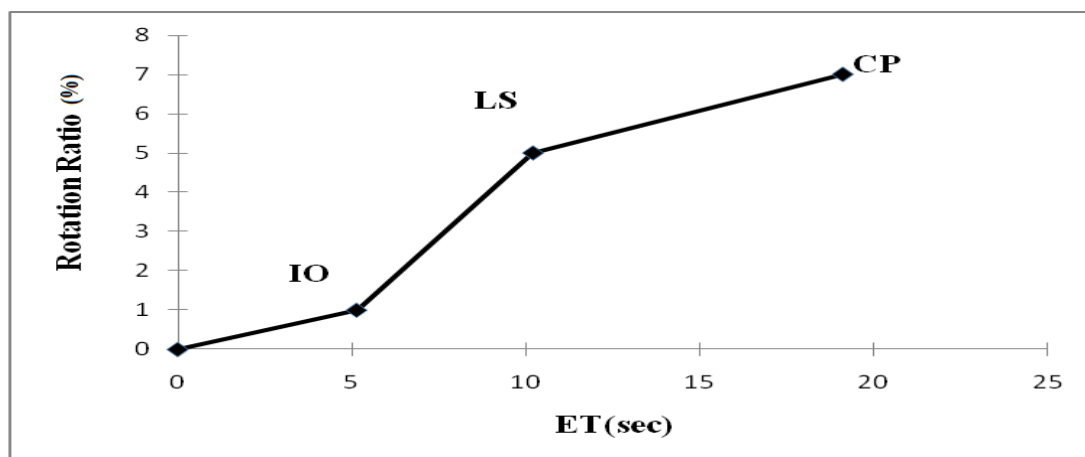


Figure 3.26 : Average Performance curve for Plastic Rotation

3.6 Results of the Jacket Analysis

3.6.1 ET Non-linear Time History Analysis Output Data

According to the fact that all ETAFs in each set are alike, the following pairs are used respectively. The first pair of ET excitations including ETA20f01 in the X direction and ETA20f02 in the Y direction, the second pair including a combination of ETA20f02 in the X direction and ETA20f03 in the Y direction, and the third pair made up of ETA20f03 and ETA20f01 in the X and Y directions. These pairs are applied to the structure alternatively, and results are averaged for final evaluation.

3.6.1.1 X-Direction Data Obtained by ETA20f

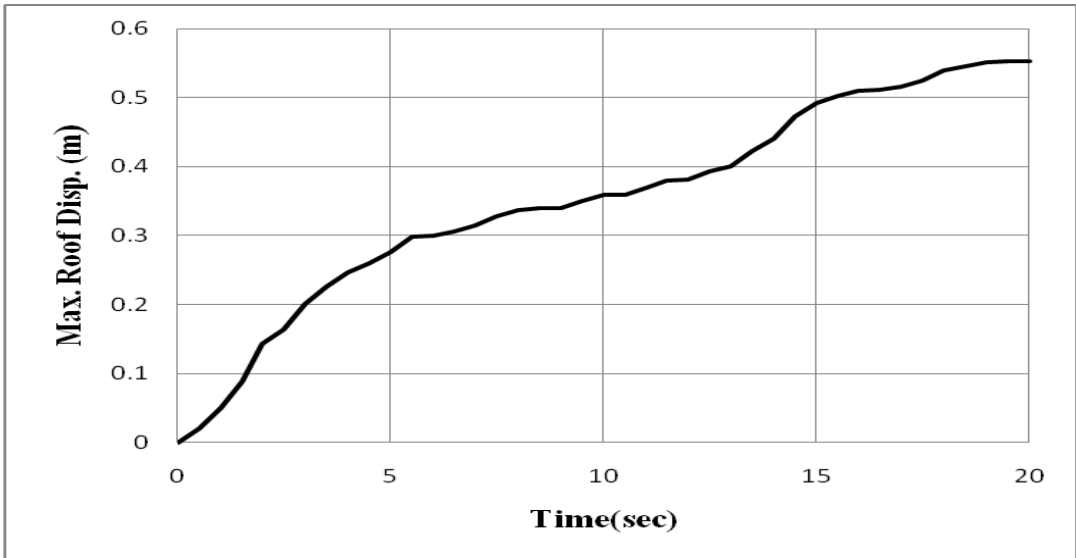


Figure 3.27 : Maximum Roof Displacement in the X-Direction Obtained by TA20f

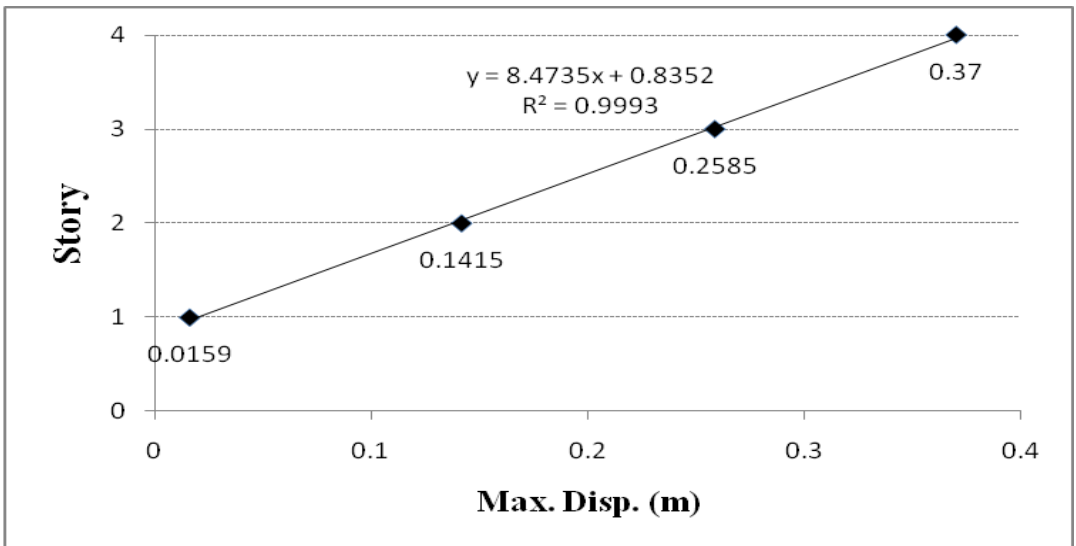


Figure 3.28 : Maximum Displacement in the X-Direction Obtained by ETA20f

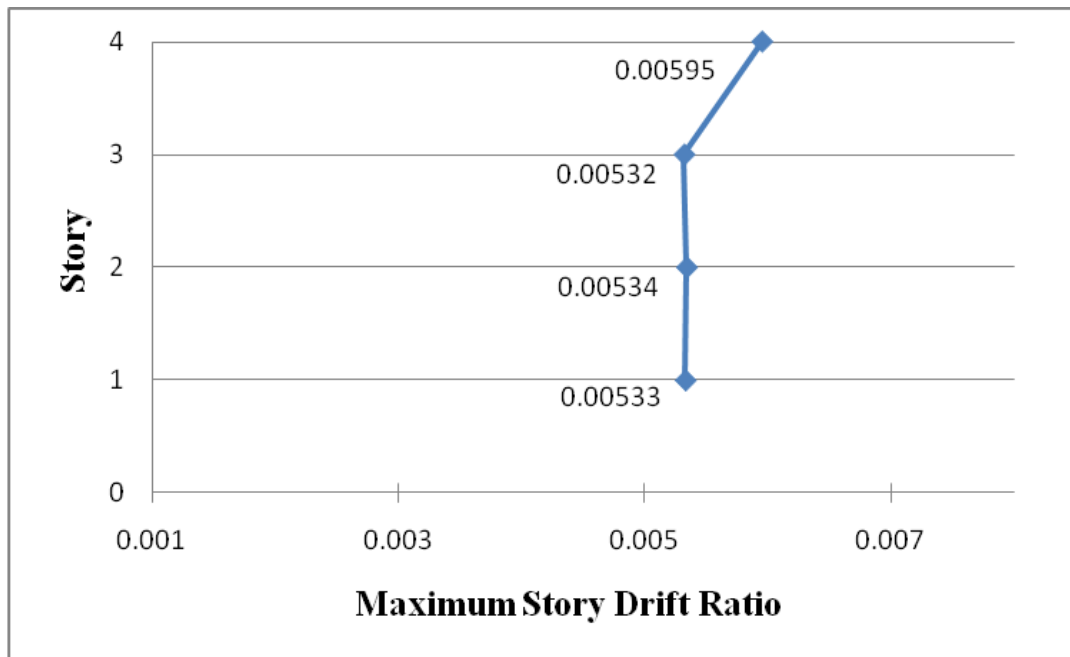


Figure 3.29 : Maximum Story Drift Ratio in the X-Direction Obtained by ETA20f

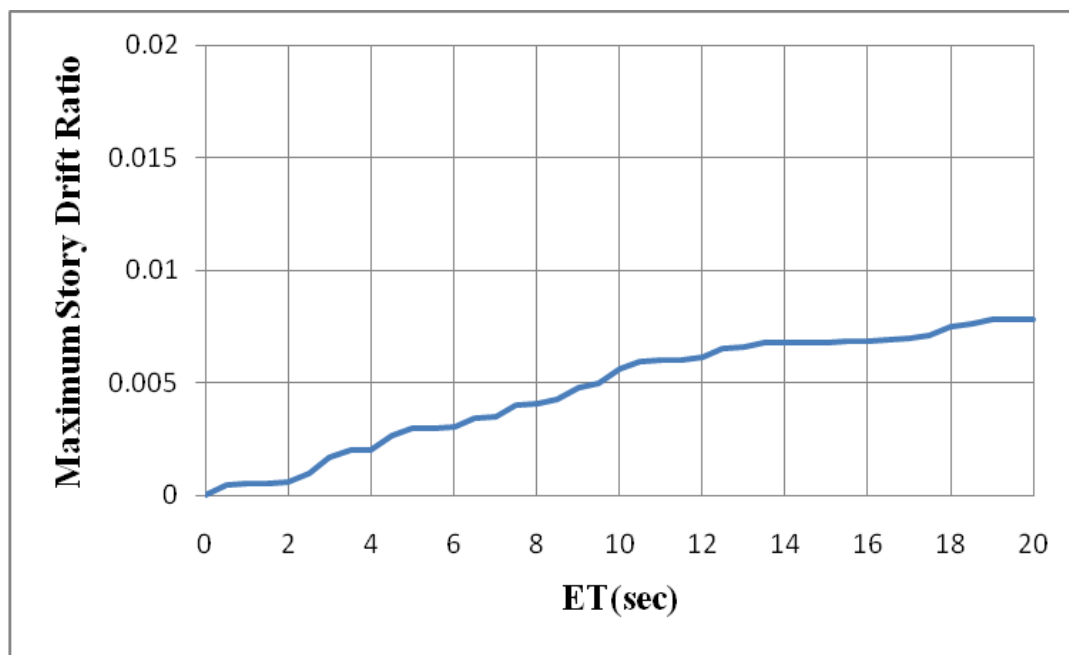


Figure 3.30 : Maximum Story Drift Ratio in the X-Direction Obtained by ETA20f

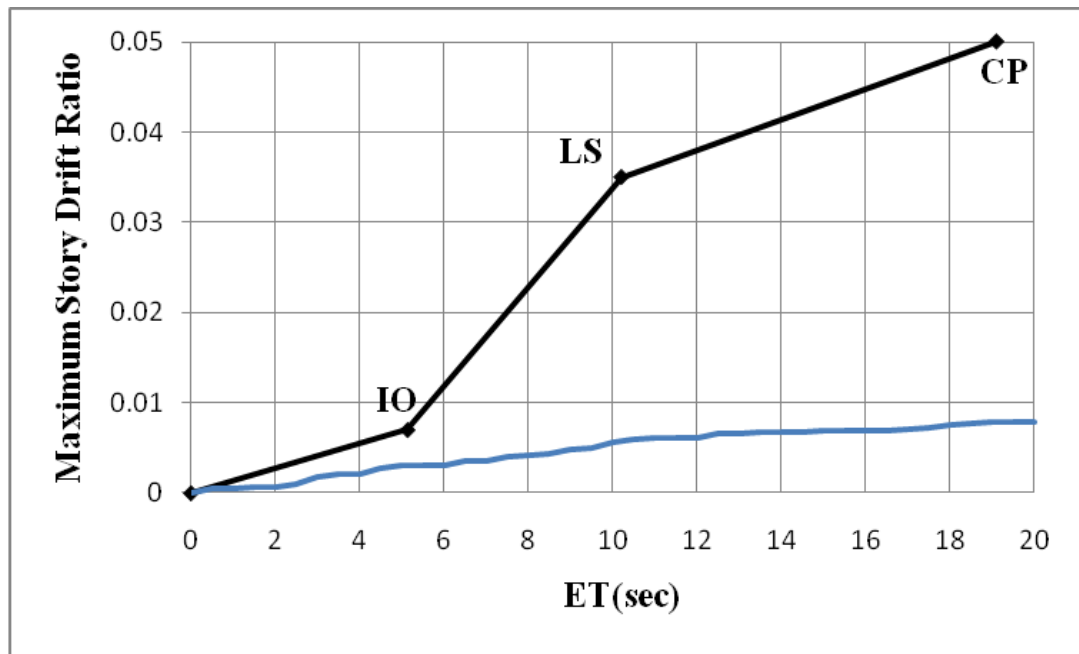


Figure 3.31 : Maximum Story Drift Ratio Compared with Reference curve

3.6.1.2 Y-Direction Data Obtained by ETA20f

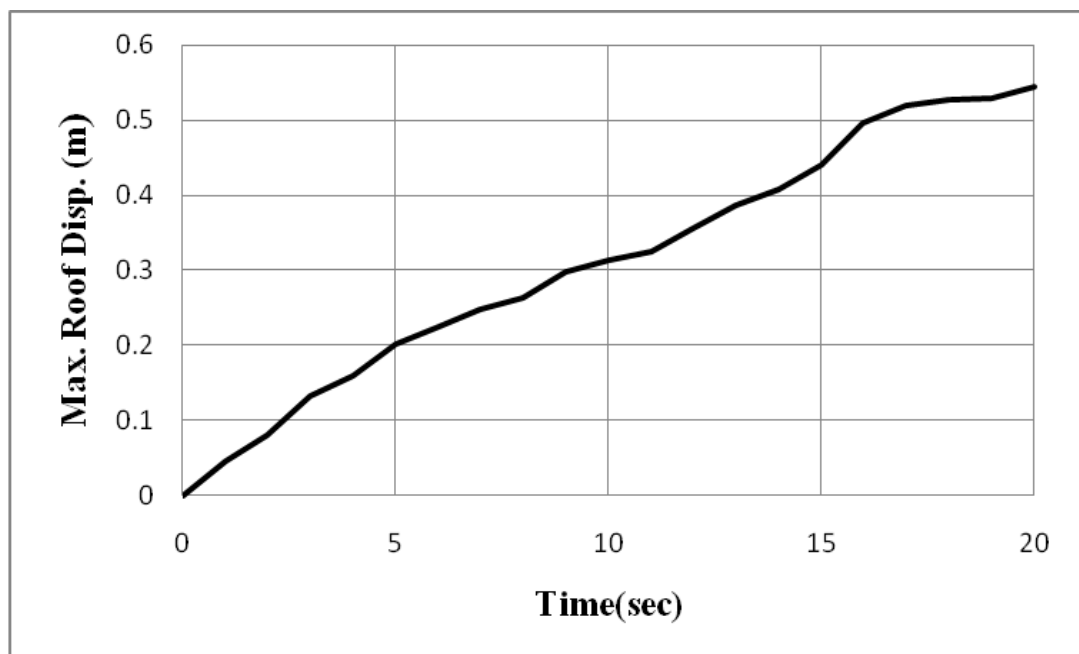


Figure 3.32 : Maximum Roof Displacement in the Y-Direction Obtained by ETA20f

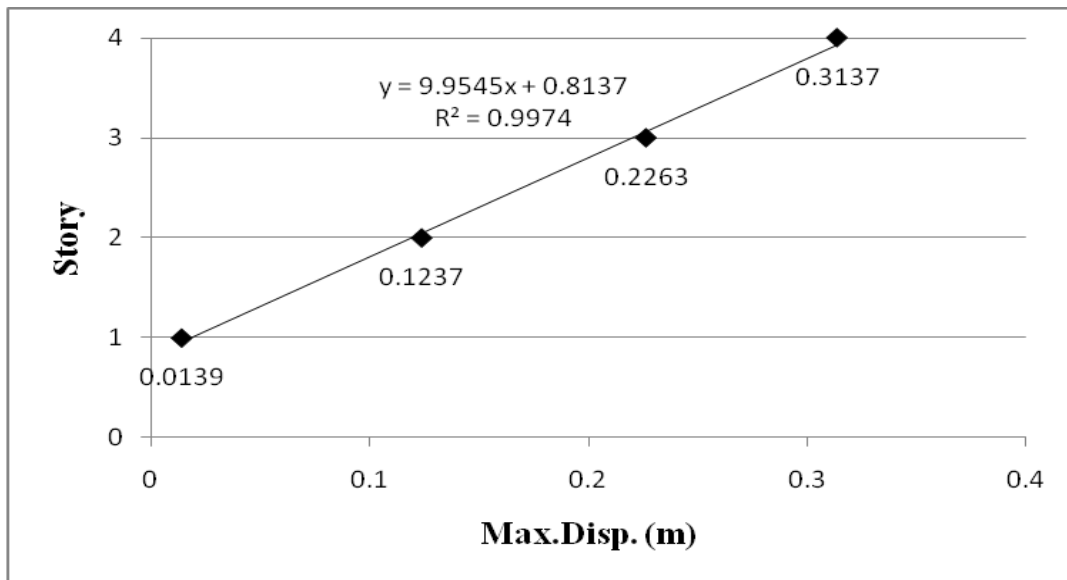


Figure 3.33 : Maximum Displacement in the Y-Direction Obtained by ETA20f

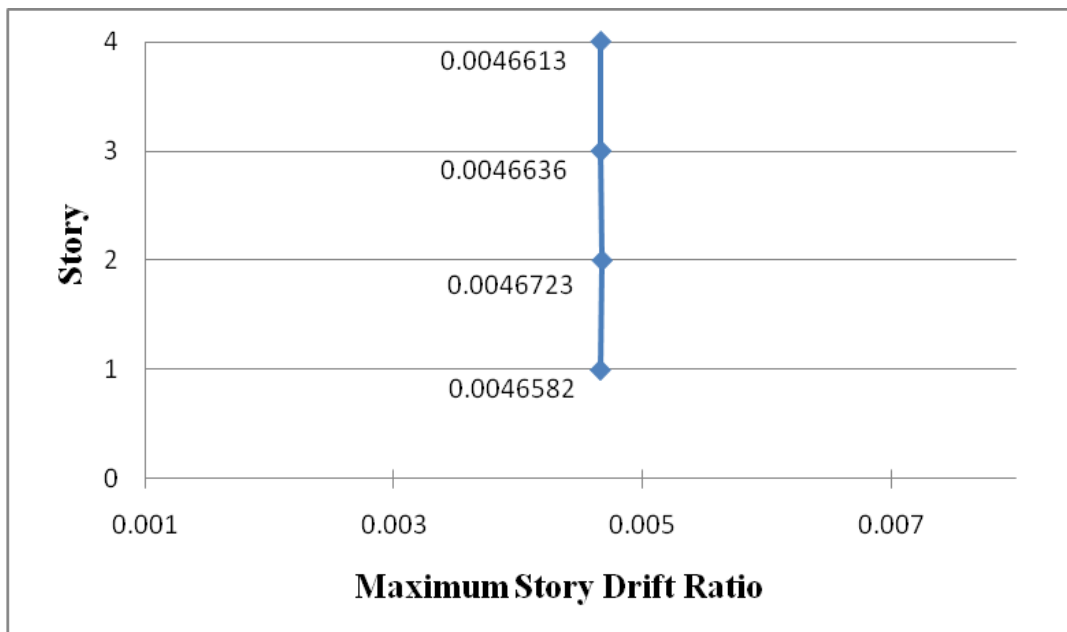


Figure 3.34 : Maximum Story Drift Ratio in the Y-Direction Obtained by ETA20f

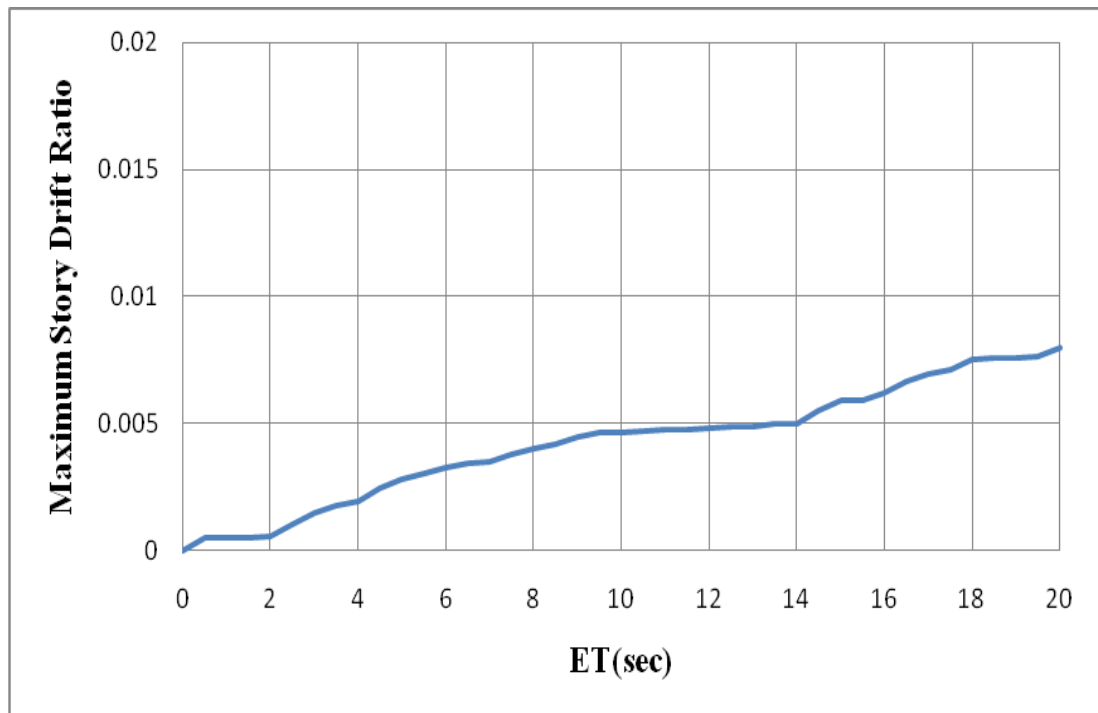


Figure 3.35 : Maximum Story Drift Ratio in the Y-Direction Obtained by ETA20f

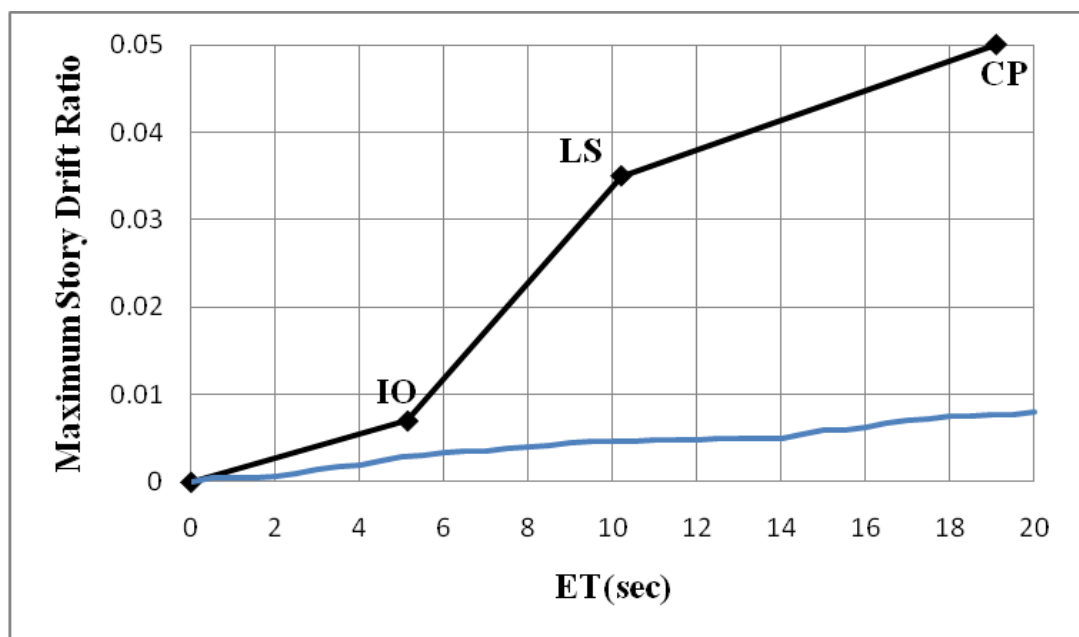


Figure 3.36 : Maximum Story Drift Ratio Compared with Reference curve

3.6.2 SPD25 Jacket Non-linear Time History Analysis Output Data under Seven Modified Accelerations

In order to evaluate the accuracy of the ET results, the jacket performance is obtained under the mentioned seven earthquake accelerations. According to the discussion in section 3.5, there are certain return periods for each relative performance level and each earthquake has a certain PGA, which mentioned in relative Table in section 3.5.

To have a good evaluation, seven earthquake records have been scaled in such a way as to create seven modified records at each performance level. To do so, some Correction Factors (CF) are required to be computed and applied to each record. This value is obtained by division of each performance level earthquake PGA to the considered acceleration PGA, e.g. the PGA of LPAND270 acceleration is 0.244g and the relative earthquake PGA of LS performance level is 0.38g, so the appropriate CF for LPAND270 will be 1.577 which shall be multiply to whole acceleration. Such a procedure shall be done for all seven accelerations for each performance level that is resulting 21 accelerations. The PGA values and acceleration CF are shown in Table 4.1

Table 3.9 : Properties of Used Records and the CF of Seven Accelerations

	LADSP 000	MHG0 6090	LPLOB 000	LPSTG 000	LPGIL 067	LPAND 270	NRORR 360
PGA	0.171g	0.259g	0.438g	0.306g	0.357g	0.244g	0.514g
CF for IO	1.053	0.695	0.411	0.588	0.504	0.615	0.35
CF for LS	2.222	1.467	0.868	1.242	0.504	1.577	0.739
CF for CP	4.035	2.664	1.575	2.255	1.933	2.828	1.342

To provide a logical overview of the performance of the jacket, it is recommended that the maximum of the drift ratio index in each PL be considered and compared to those related ET analysis results. This procedure is done by SAP2000 Software and maximum of drift ratio is calculated. The following figures are the drift values of the jacket in X-direction when the structure behavior is in nonlinear range. To reduce the repetitive procedures, the Y-direction figures are not mentioned here but the procedure is the same as X-direction.

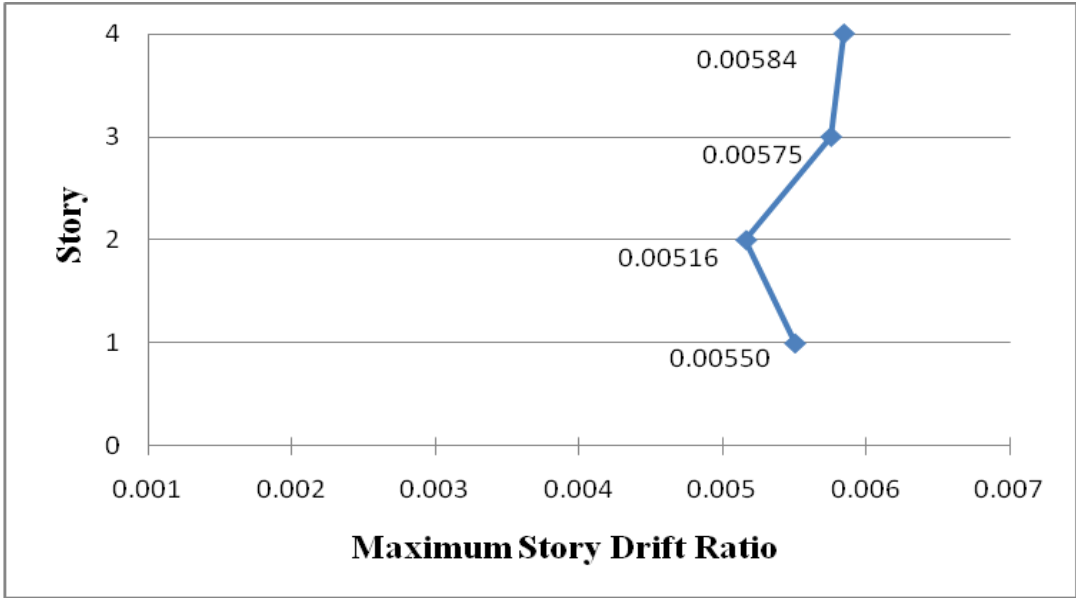


Figure 3.37 : Maximum Story Drift Ratio Obtained by LADSP Record

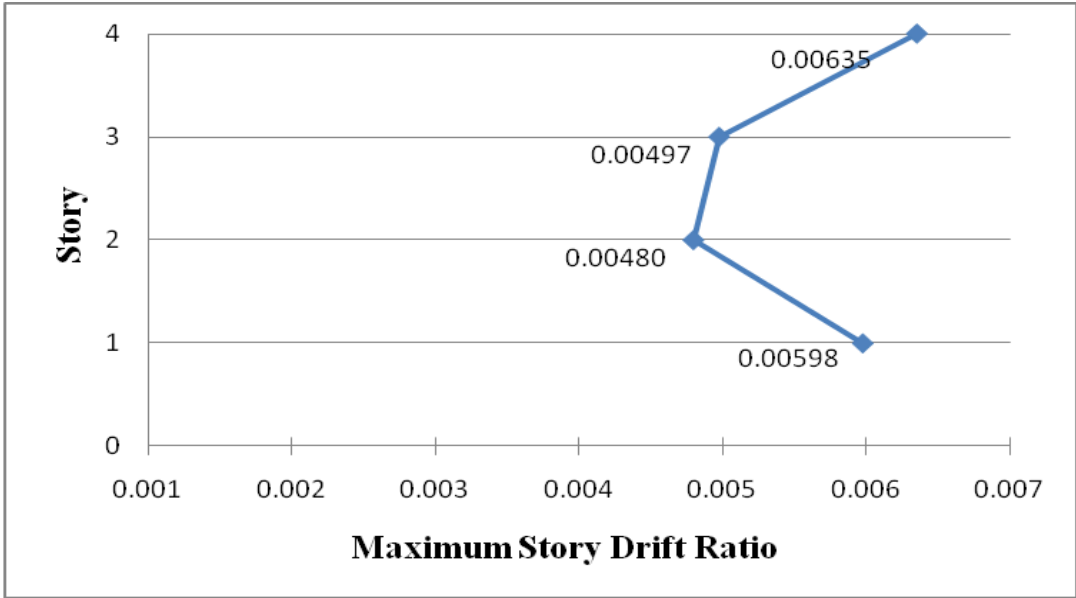


Figure 3.38 : Maximum Story Drift Ratio Obtained by LPAND Record

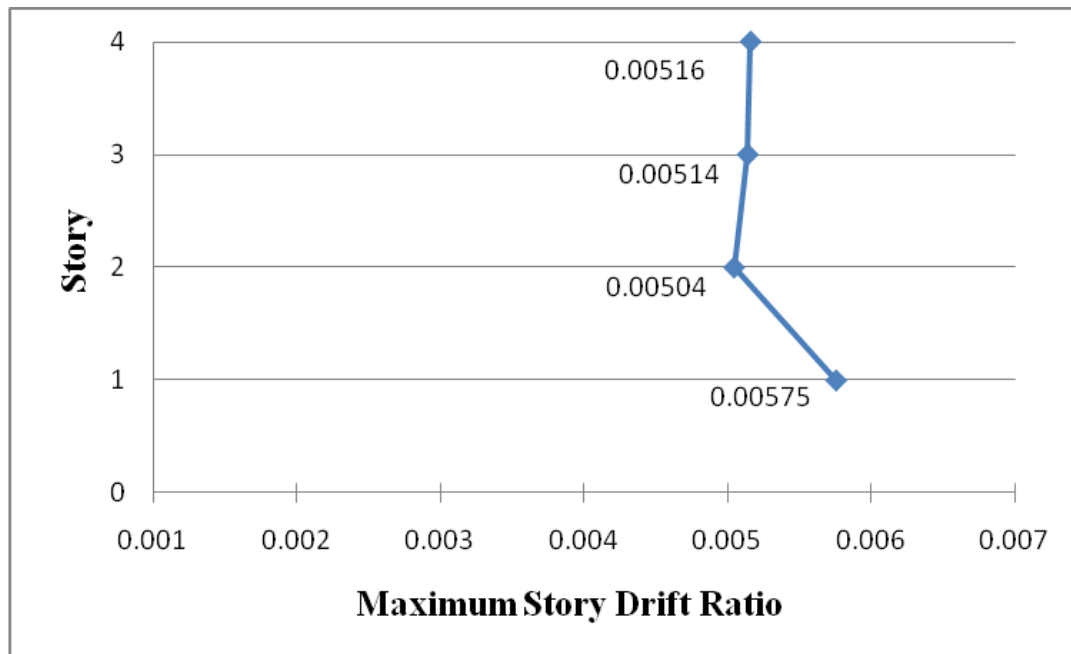


Figure 3.39 : Maximum Story Drift Ratio Obtained by LPGIL Record

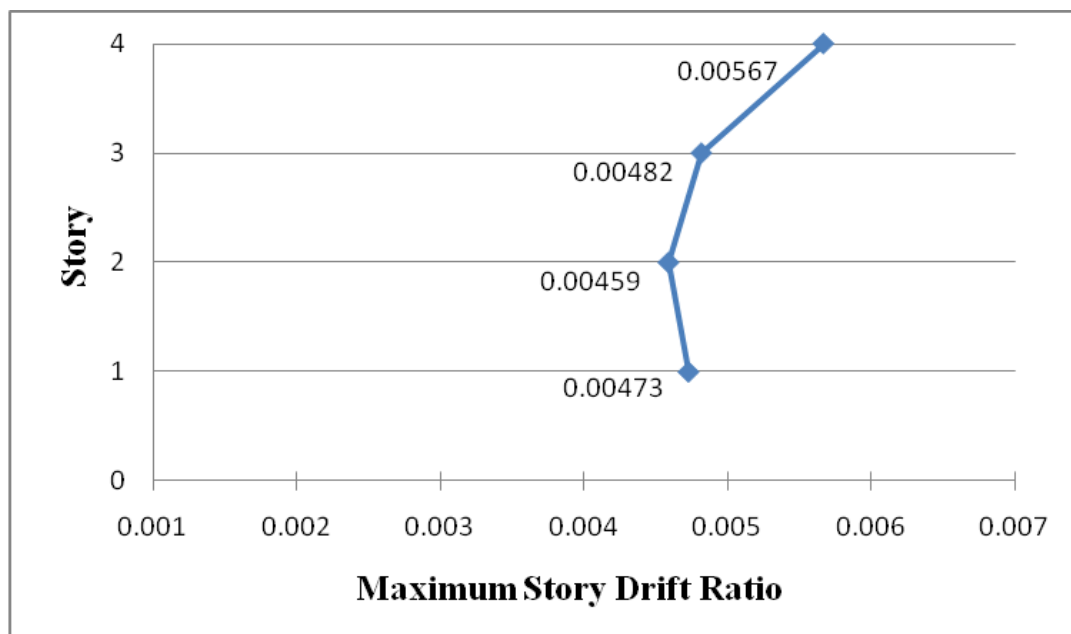


Figure 3.40 : Maximum Story Drift Ratio Obtained by LPLOB Record

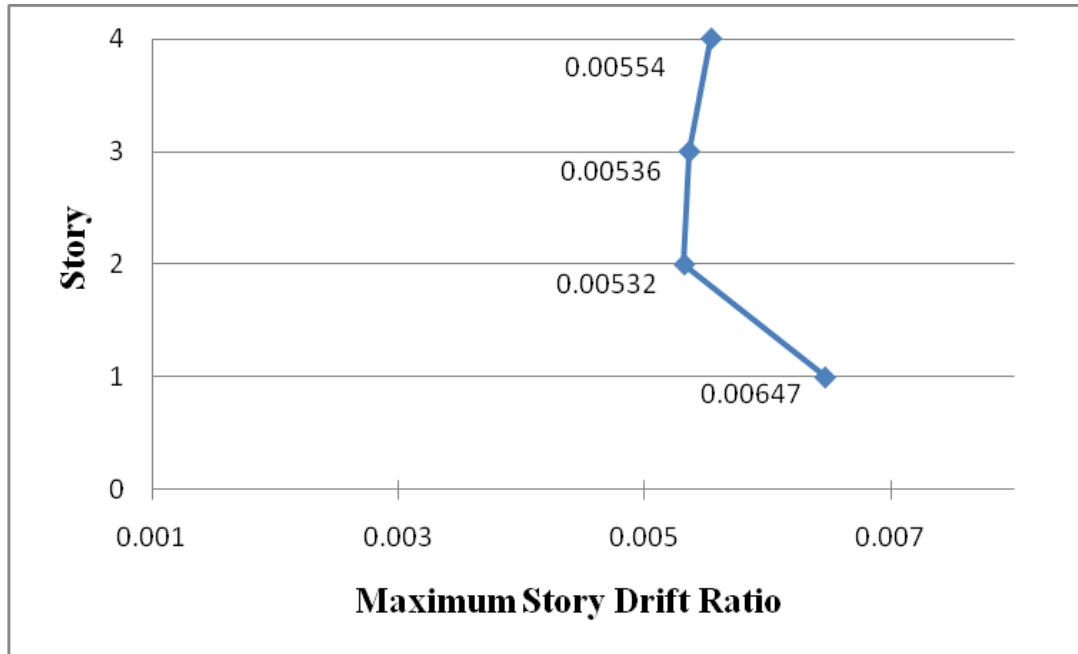


Figure 3.41 : Maximum Story Drift Ratio Obtained by LPSTG Record

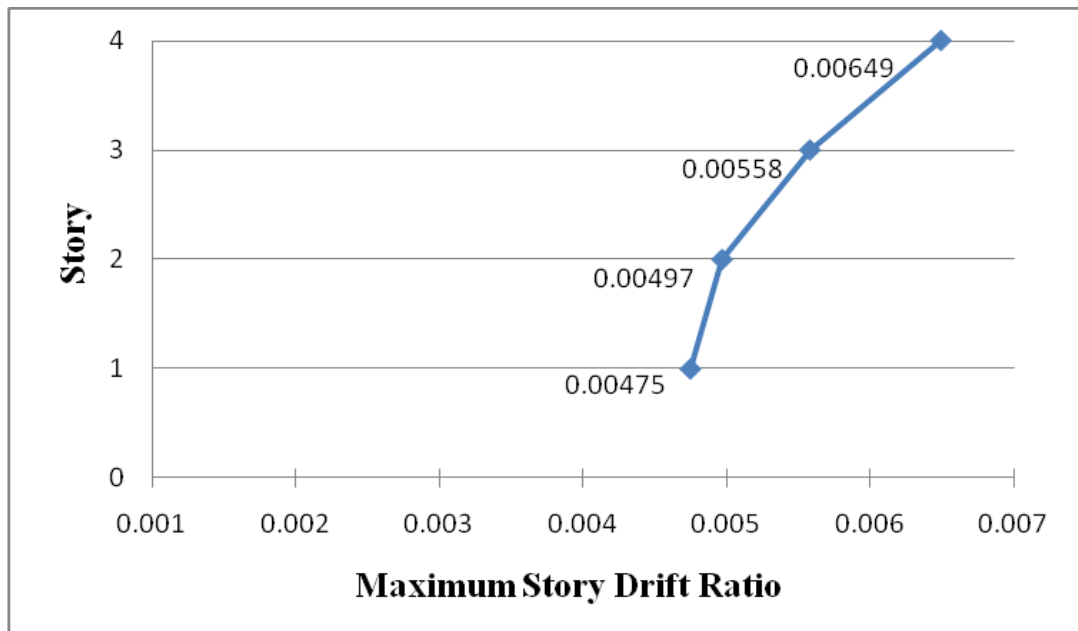


Figure 3.42 : Maximum Story Drift Ratio Obtained by MHGO Record

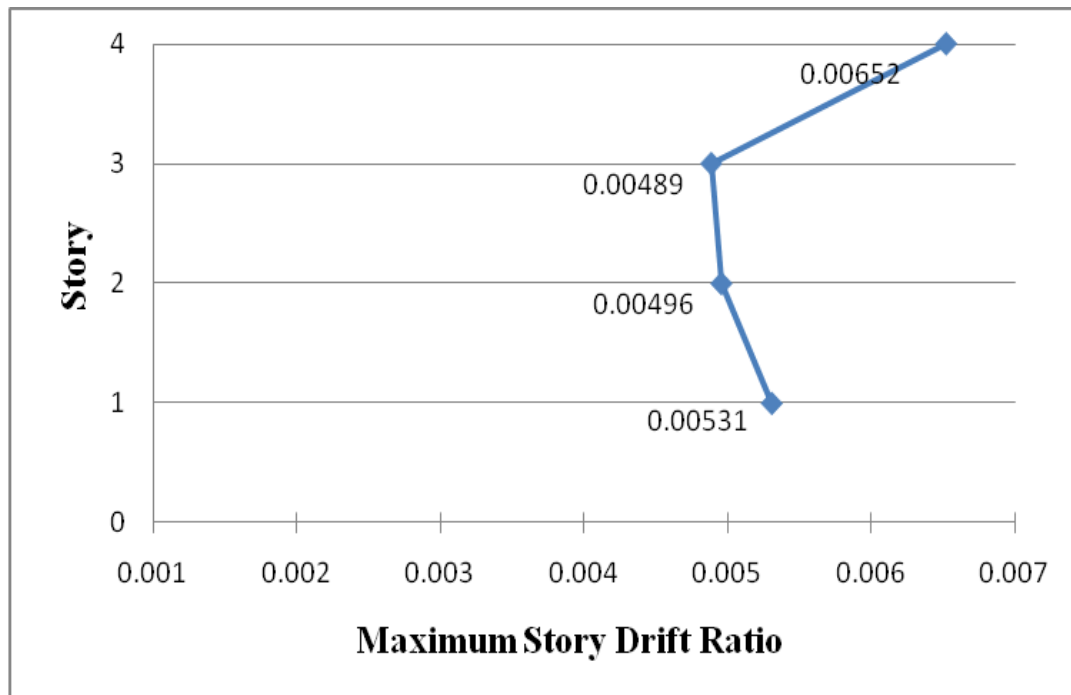


Figure 3.43 : Maximum Story Drift Ratio Obtained by NRORR Record

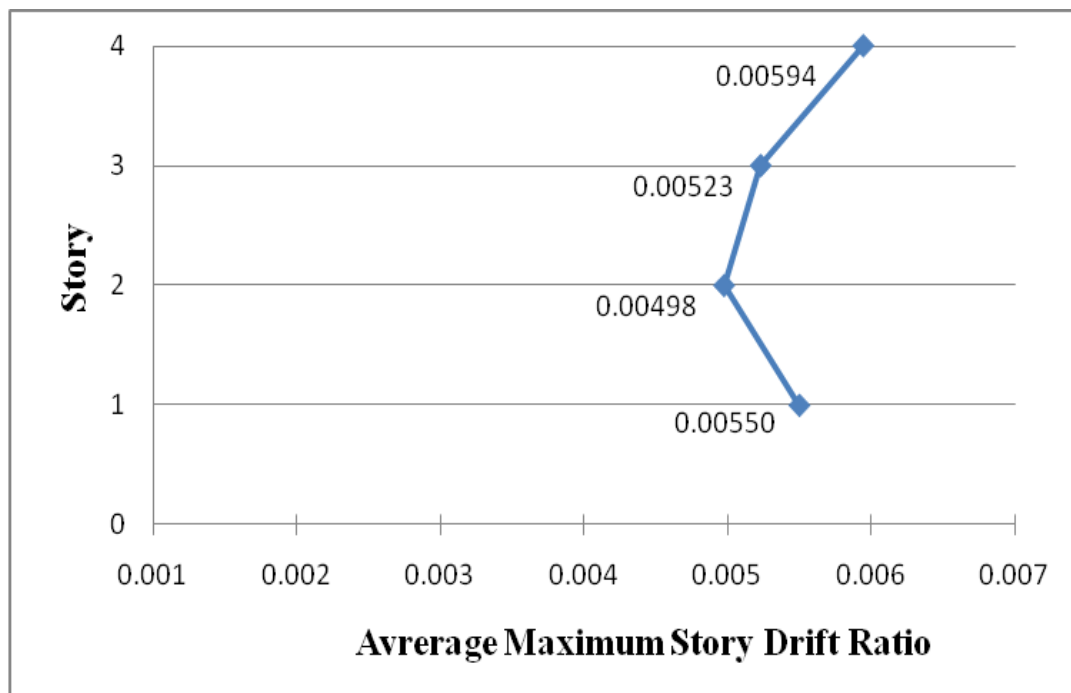


Figure 3.44 : Average Maximum Story Drift Ratio of Seven Modified Records

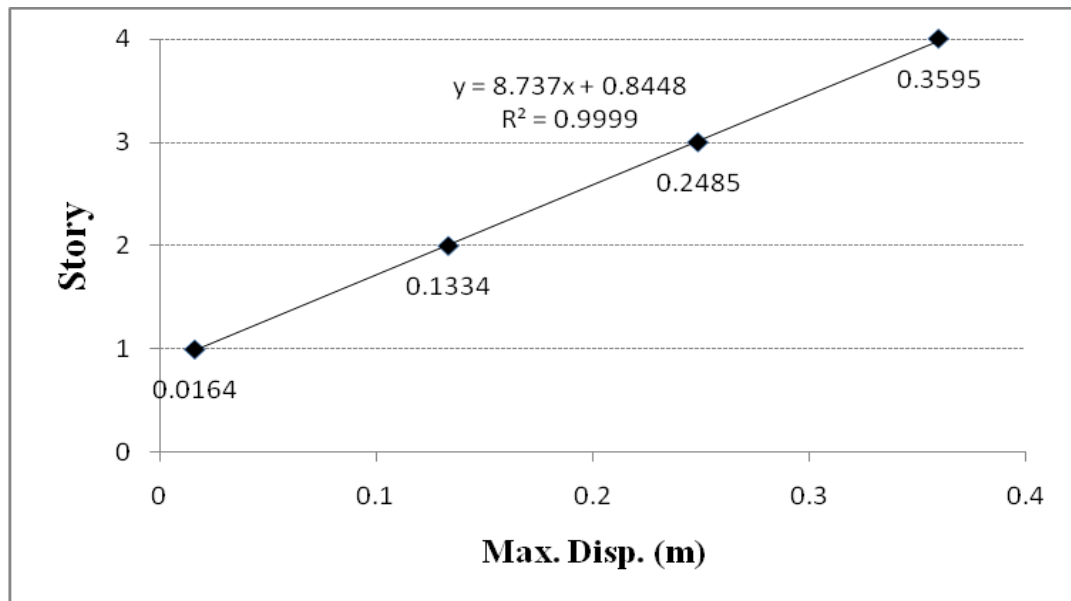


Figure 3.45 : Average Maximum Displacement in the X-Direction Obtained by Seven Modified Records

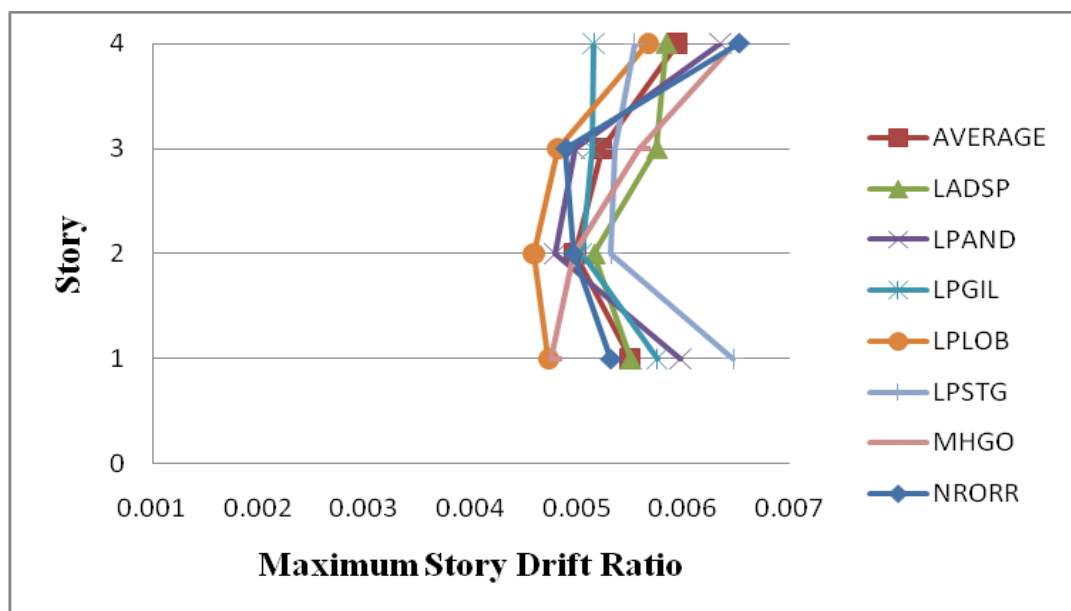


Figure 3.46 : Average and 7 Records Max. Displacement in the X-Direction

Table 3.10 : Max. Drift Ratio Table for Seven Time History Analyses

Record	Max. Drift Ratio			
	Story			
	1	2	3	4
AVERAGE	0.00550	0.00498	0.00523	0.00594
LADSP	0.00550	0.00516	0.00575	0.00584
LPAND	0.00598	0.00480	0.00497	0.00635
LPGIL	0.00575	0.00504	0.00514	0.00516
LPLOB	0.00473	0.00459	0.00482	0.00567
LPSTG	0.00647	0.00532	0.00536	0.00554
MHGO	0.00475	0.00497	0.00558	0.00649
NRORR	0.00531	0.00496	0.00489	0.00652

3.6.2.1 Nonlinear Static Pushover Analysis Output Data

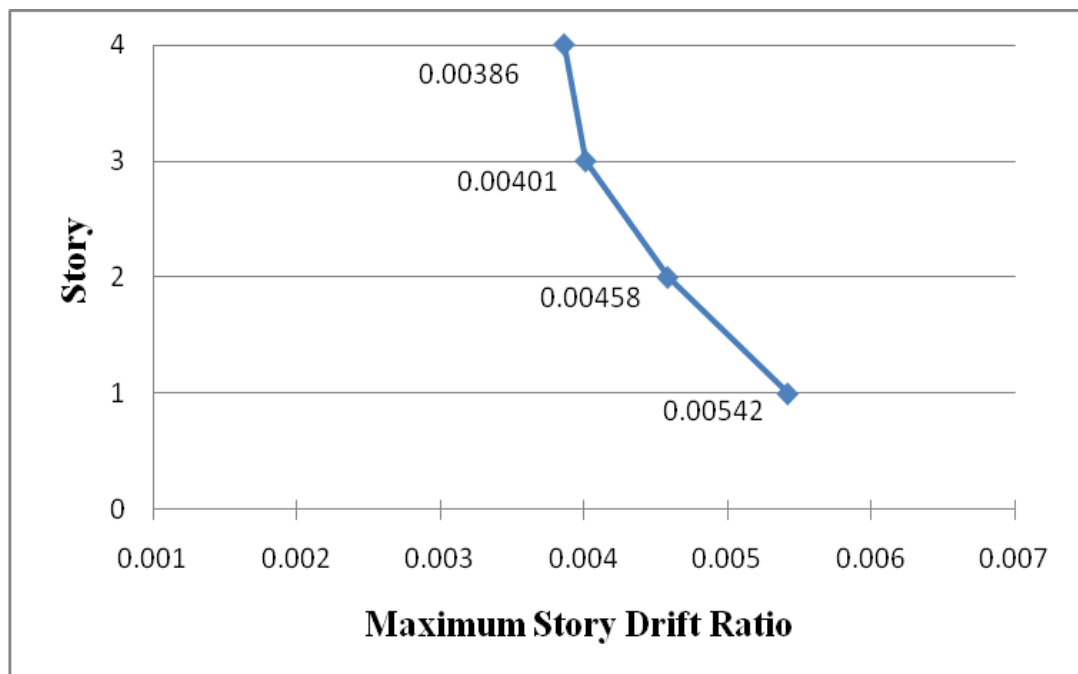


Figure 3.47 : Maximum Drift Ratio in the X-Direction Obtained by Pushover

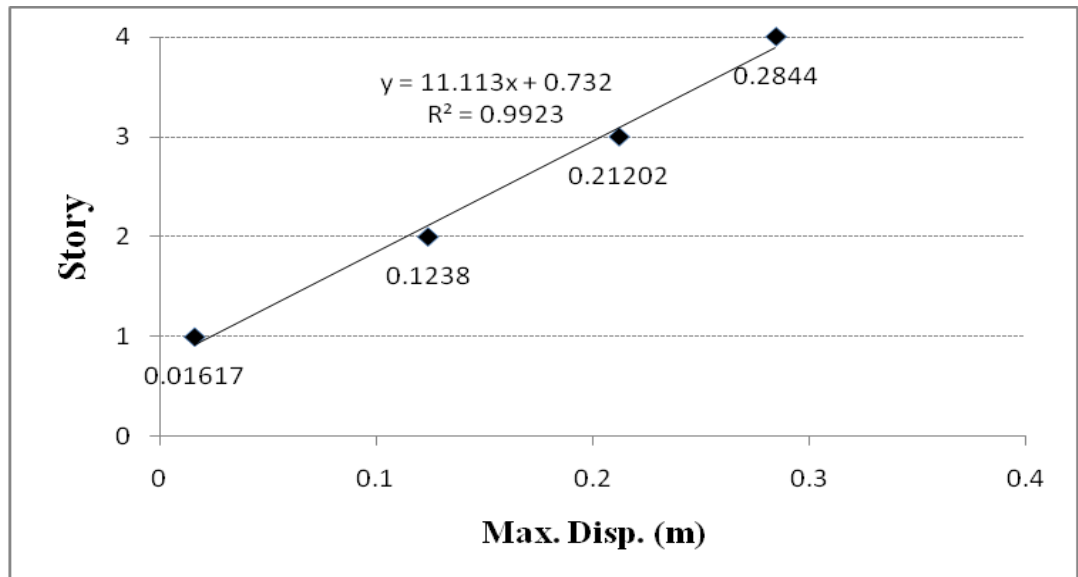


Figure 3.48 : Maximum Displacement in the X-Direction Obtained by Pushover

3.6.2.2 Modal Analysis Output Data

Thirty-five modes were extracted to obtain a cumulative mass participation factor of more than 95% in the global X and Y direction. Table below tabulates the period, mass participation factor for the mode shapes generated for the jacket. The mode shape plots are as shown in Appendix C.

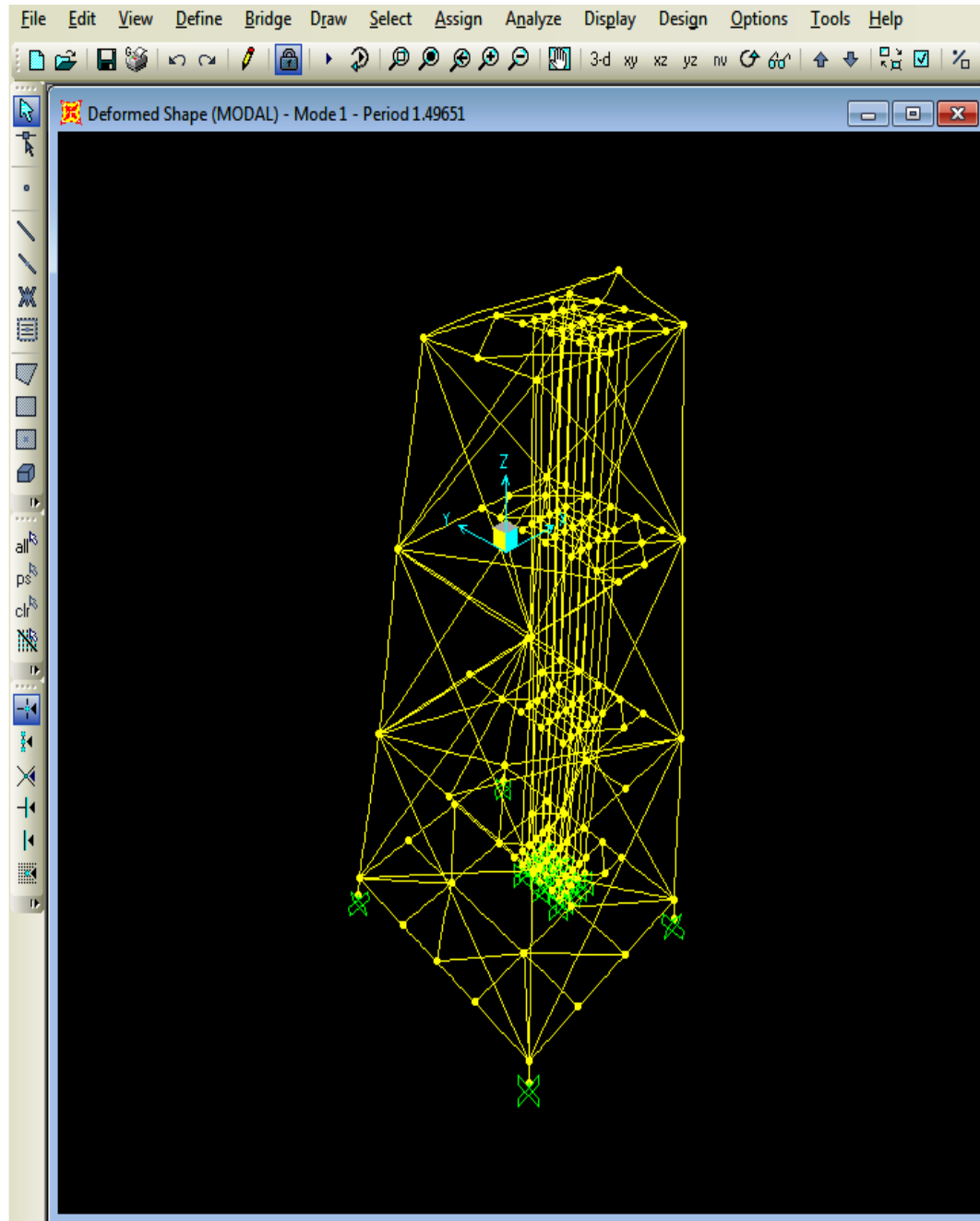


Figure 3.49 : First Mode of Modal Analysis

Table 3.11 : Period and Mass Participation Factor Summary

Mode	Period (s)	Mode Shape	Cumulative Factor		
			X	Y	Z
1	1.4965	1st Global X bending	0.7414	0.0175	0.0006
2	1.477	1st Global Y bending	0.7743	0.6156	0.0006
3	1.1690	1st Global Torsion	0.7810	0.8071	0.0007
4	1.108	2nd Global Y bending	0.7845	0.9644	0.0028
5	0.410	3rd Global Y bending	0.9767	0.9661	0.0125
6	0.40	2nd Global Torsion	0.9781	0.9694	0.0129
7	0.39	-	0.9788	0.9849	0.1225
8	0.38	-	0.9790	0.9916	0.2716
9	0.36	-	0.9793	0.9916	0.2823
10	0.352	-	0.9905	0.9917	0.2977
11	0.33	-	0.9979	0.9917	0.7007
12	0.28	-	0.9979	0.9917	0.8054
13	0.27	-	0.9979	0.9917	0.8115
14	0.27	-	0.9979	0.9927	0.8188
15	0.26	-	0.9979	0.9928	0.9168
16	0.26	-	0.9979	0.9934	0.9187
17	0.25	-	0.9979	0.9979	0.9194
18	0.24	-	0.9980	0.9979	0.9437
19	0.21	-	0.9980	0.9982	0.9552
20	0.19	-	0.9981	0.9987	0.9569
21	0.19	-	0.9981	0.9987	0.9570
22	0.18	-	0.9991	0.9987	0.9571

23	0.18	-	0.9993	0.9989	0.9573
24	0.16	-	0.9993	0.9989	0.9582
25	0.15	-	0.9993	0.9989	0.9588
26	0.14	-	0.9993	0.9989	0.9590
27	0.14	-	0.9993	0.9995	0.9590
28	0.13	-	0.9995	0.9995	0.9600
29	0.13	-	0.9995	0.9995	0.9601
30	0.13	-	0.9995	0.9995	0.9606
31	0.12	-	0.9995	0.9995	0.9630
32	0.12	-	0.9995	0.9995	0.9635
33	0.12	-	0.9995	0.9996	0.9688
34	0.12	-	0.9996	0.9996	0.9689
35	0.11	-	0.9996	0.9996	0.9719

3.6.3 Result Comparison

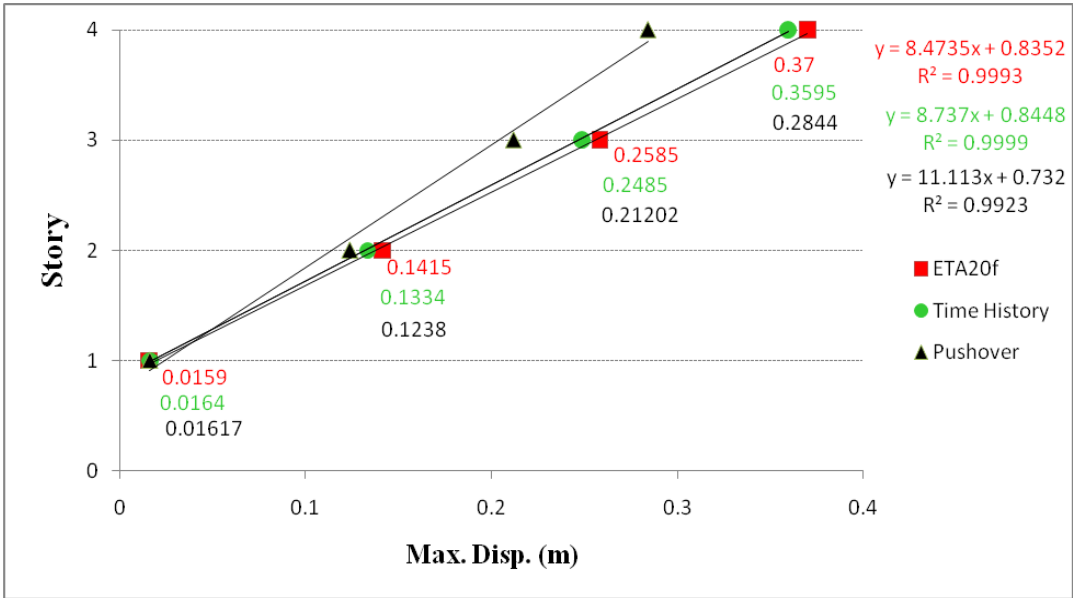


Figure 3.50 : Compared Maximum Story Displacement in X-Direction

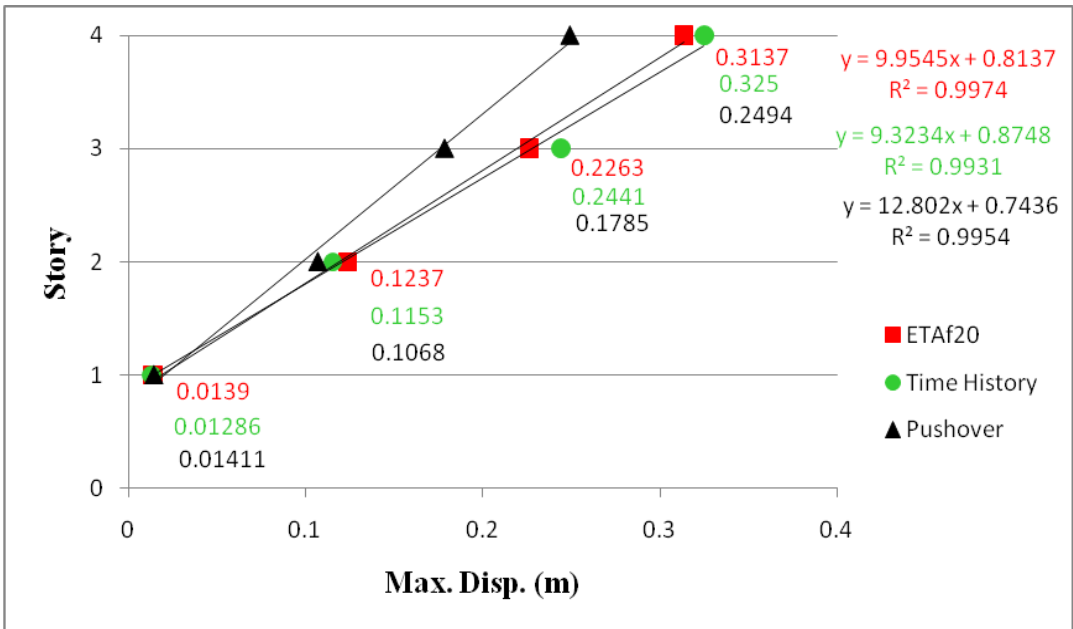


Figure 3.51 : Compared Maximum Story Displacement in Y-Direction

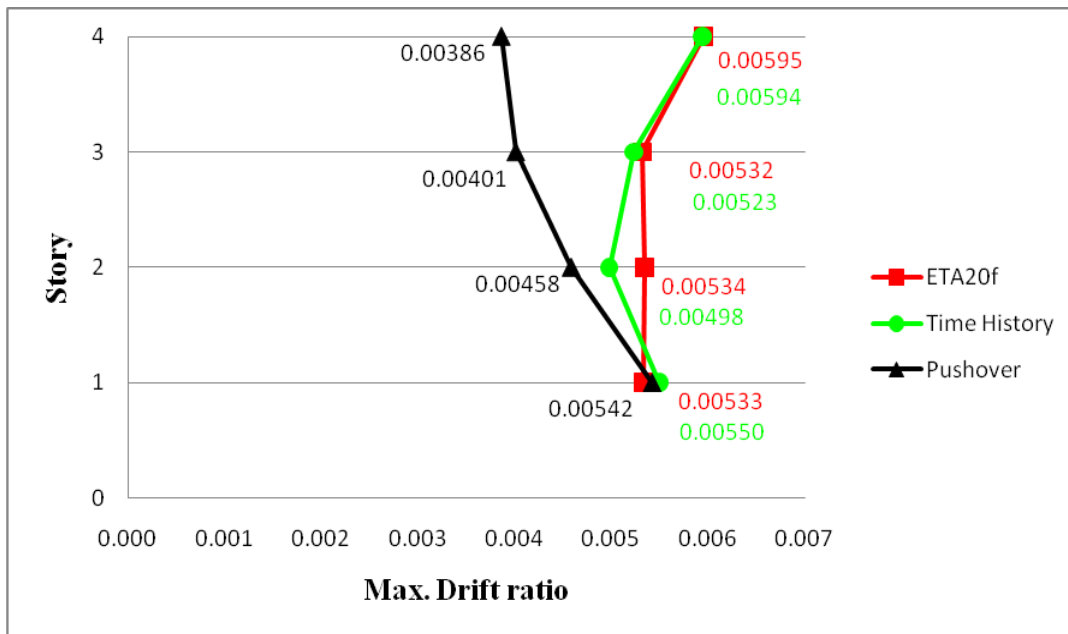


Figure 3.52 : Compared Maximum Drift Ratio in X-Direction

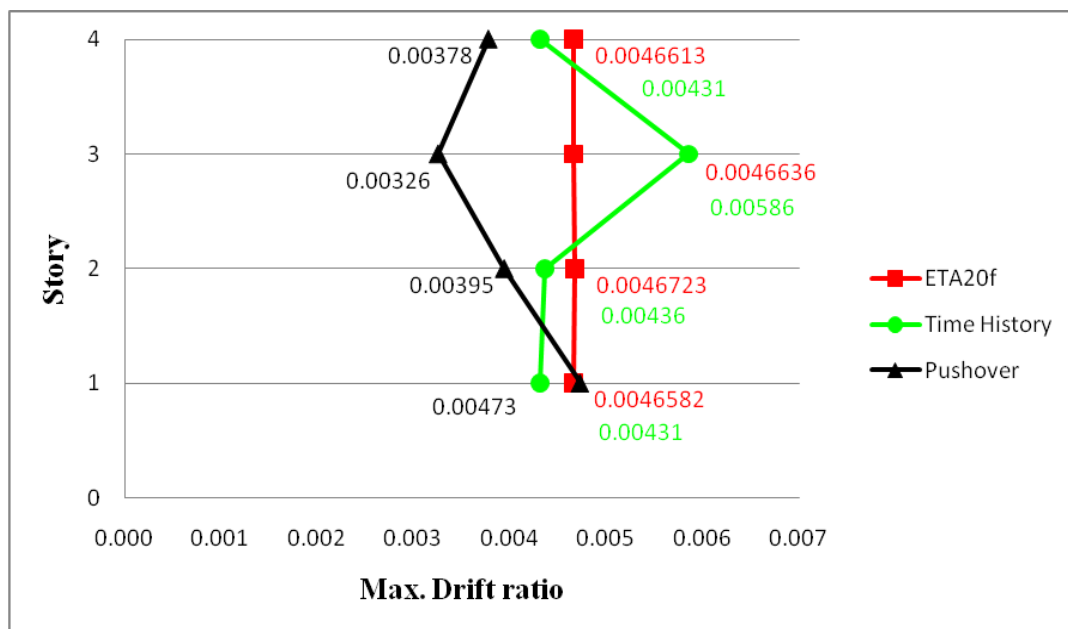


Figure 3.53 : Compared Maximum Drift Ratio in Y-Direction

4. CONCLUSION AND RECOMMENDATIONS

Conclusion:

In this thesis, a procedure for extending the ET method to the jacket structure is proposed and the validity and the applicability of the ET method in the nonlinear seismic analysis are assessed. Since, the story drift has influential effects on nonlinear structural elements and is considered as an important damage criterion in design code, the discussion focus is on the maximum story drift ratio and maximum roof displacement of jacket. This goal is obtained by comparing the results of ET analysis, pushover analysis and time-history analysis.

In the proposed procedure for the jacket analysis by the ET method, the structure is analyzed under intensifying ETAFs applied in principal directions of the structure simultaneously, and response of the structure is monitored versus time. Then the value of responses, such as maximum drift is checked at a target time (or times) when the intensity excitation matches the particular design level excitation.

The case study jacket, SPD25, is analyzed with the mentioned approaches under bidirectional excitations according to seismic codes, and results were investigated and shown as following:

- 1) The main advantage of the ET method over the regular time-history and pushover methods is its small number of analysis needed.
- 2) As shown in figures (Figure 3.49 to 52), all methods have almost same results in the first story but the results will vary in upper stories especially for maximum story displacement by pushover analysis, and the story drift ratio results change in third story but will be closed to each other in forth story. The average results from time history analysis using ground motions in bidirectional can be estimated by the ET method with acceptable approximation but the pushover analysis have some differences.

It is clear that the results from Time-History analysis are the exact one, and the ET results are near to these results but the main reason for the difference between the results of the pushover analysis and those other two methods is the invariant load pattern used for the pushover analysis. The load pattern in pushover analysis is based on just one mode of vibration and it does not consider the contribution of higher modes. Moreover, the pushover analysis is the semi-dynamic analysis so; it is obvious that the results have differences with those two time-history base analysis.

Of course, using advanced procedure such as modal pushover analysis or the adaptive modal combination method will improve the results of the pushover analysis.

- 3) Since, the seismic intensifying in ET method cannot cause the members to be in plastic phase in early seconds, so, the plastic rotation of the elements shall be zero, and in this range, the story drift ratio will be the determination parameter for the structure performance. This time range will increase in tall structures such as this jacket, because the members are designed stronger than ordinary buildings. So the members will be plastic in the higher times, it means that the structure endurance time is increased. (Figure 3.31 & 3.36)
- 4) The defined range and limit by FEMA356 for story drift ratio seems to be good for IO performance level according to the obtained results. But, since, we assumed if 'f' series of accelerograms were applied to the structure it should be capable of meeting the 'IO' performance level up to 5.14 sec, the LS performance level up to 10.21 sec, and the CP performance level up to 19.11 sec, (Figure 3.24). So, there are four interpretations:

First, as mentioned before, the procedure to obtain the PGAs is not exact one and to have an exact calculation, it is needed to obtain the hazard curve for a special site and the PGA should be acquired according to that curve.

Second, the ET accelerograms should be modified; since these accelerograms are calibrated by FEMA Code spectrum, it seems not to be good way.

Third, the suggested drifts by FEMA are general. To specify the drifts for marine structures, we should reference to the API, but the API has no limitation for story drift ratio because the design principle of this code is based on the member stress not structure performance, and here we analyze the structure rather than designee. Therefore, according to the thesis analysis methods, the jacket member's stresses can be obtained and compared with API or new drifts can be defined to improve the standard, which this subject can be suggested for PhD research.

Fourth, the structure may be designed stronger that the jacket IO performance is in second 10th instead of second 5th.

However, based on the experts knowledge in this field, the jacket structures should even endure more than this time, so I think the best suggestion is the third one.

Recommendations:

For further research in this field, the below recommendations are suggested:

- 1) In this study, the 'f' series of accelerograms are used to obtain the performance of the jacket. The other compatible non-linear accelerograms, which exist in the Sharif Technical University website, can be used for better evaluation of structure performance and ET compatibility with Time history analysis.
- 2) In this project, the jacket structure is studied at target time second 10, (IO performance level), for further research, the LS and CP excitation level can be studied.
- 3) In this thesis, the story drift ratio is investigated. In order to study members performance and finally structure performance, the members internal force, stresses and rotation can be studied.
- 4) According to some existing articles, Damage Level index is introduced for structure that the performance can be easily evaluated. In fact, in order to evaluate the structure performance, the critical drift ratio and plastic rotation and other parameters should be considered, that is practically is difficult while, the Damage Level index can illustrate the structure performance easily. For further research, this index can be introduced to the jacket structure.
- 5) In this thesis, because of some social limitation, just one jacket is studied. in order to improve the results other jacket can be investigated for further research. It is noted that the jacket height and members dimensions will effect on the results.

REFERENCES

- [1] **Krawinkler, H. and Miranda, E., 2004** "Performance-based earthquake engineering", in Earthquake engineering: From engineering Seismology to Performance-Based engineering. Y.Bozorgian and V.V. Bertero, Eds. CRC Press, US A
- [2] **V.Valamanesh, H.E.estekanchi, A.Vafai, M.Ghaemian, 2011** "Application of the endurance time method in seismic analysis of concrete gravity dams" Scientica Iranica, 18(3), 326-337accepted24Aprill 2011 from <http://sciencedirect.com>
- [3] **FEMA-302, 1997** "NEHRP recommended provisions for seismic regulations for new buildings and other structures". Part 1-Provisions prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency. Washington, D.C
- [4] **FEMA-273, 1997** "NEHRP guidelines for the seismic rehabilitation of buildings" FEMA-273 Federal Emergency Management Agency, Washington, D.C
- [5] **A.Mirzaee, H.E.Estekanchi, and A. Vafai, 2010** "Application of endurance Time Method in Performance-based Design of Steel moment Frames" Scientica Iranica, vol. 17 No.6, pp. 482 492
- [6] **V. Vallamanesh, H.E. Estekanchi, and A. Vafai, 2005** "Evaluation of Endurance Time method in the analysis of 2D steel frames, in compared with the Static analysis method" No.35 in Persian language.
- [7] **Fugro Survey (Middle East) Ltd. Engineering Report South Pars Gas Field Development** Phase 17 & 18 Geotechnical Soil Borings South Pars Field offshore Iran Rev 3 April 2007
- [8] **N.D.P.Bartrop, A.J.Adams, Atkins Oil & Gas Engineering Limited, Epsom, UK, Dynamic, 1991** Third Edition "Dynamic of Fixed Marine Structures 1&2" Linacre House, Jordan Hill, Oxford OX2 8DP]page 211 VOL.1
- [9] **API API-RP2A WSD 2000 2A_WSD: Recommended Practice for Planning, Design and Constructing Fixed Offshore Platform Working Stress Design**, 21st Edition, and Dec 2000.
- [10] **NIOC South Pars Field Development Project Phases 17 & 18 Offshore Platform** Document No SP1718-P3-ST-CS-8202 REV 2
- [11] **NIOC South Pars Field Development Structural Material Specification** Document No SP17/18-P2-ST-SP-8028
- [12] **FEMA-356, 2000** "Pre-standard and commentary for the seismic rehabilitation of buildings" Federal Emergency Management Agency Washington D.C

- [13] **Ahmed Ghobarah, 2001** "Performance-based design in earthquake engineering: state of development". Engineering Structures, Vol 23, No 5, pp. 875-884
- [14] **Bozorgnia.Y Bertero.V, 2004** "Earthquake Engineering from Engineering Seismology to Performance-Based Engineering CRC PRESS
- [15] **Estekanchi, H.E., Vafai, A. and Sadeghazar,M 2004**, "Endurance time method for seismic analysis and design of structures". Scientia Iranica. Vol. 11, No. 4, pp 361-370
- [16] **Peter Fajfar, N. EERI, 2000** "A Nonlinear Analysis Method for performance Based Seismic Design", Earthquake spectra, vol 16, No 3, pp 573-592, August
- [17] **Clough R.W. and Penzien J. 1993** "Dynamics of structures", McGraw-Hill Inc
- [18] **Riahi H.T, Estekanchi HE, 2007** "Comparison of different methods for selection and scaling of ground motion time histories", 5th International conference on seismology and earthquake engineering
- [19] **FEMA-440, 2005** "Improvement of nonlinear static seismic analysis procedures", Washington DC: Federal Emergency Management Agency
- [20] **Kaila, K.L. and Narian. H. 1971**, "A new approach for the preparation of quantitative seismicity maps" Bull. Seis. Soc. Amer. 61. No. 1275-91
- [21] **Riahi H.T, Estekanchi HE, 2009** "seismic assessment of steel with the endurance time method", Journal of Construction Steel Research 66(2010)780-792, <http://elsevier.com/locate/jcsr>
- [22] **Riahi H.T, Estekanchi H.E and Vafai.A 2009** "Endurance Time Method-Application in Nonlinear Seismic Analysis of Single Degree of Freedom System" Journal of Applied Sciences 9(10): 1817-1832, ISSN 1812-5654, Asian Network for Scientific Information.
- [23] **V.Valamanesh, H.E.estekanchi, 2010** "Compatibility of the endurance time method with codified seismic analysis approaches on three-dimensional analysis of steel frames" Structural Design tall Design of Tall Special Buildings, published online in Wiley Online Library <http://wileyonlinelibrary.com>, DOI:10.1002/tal.666

Url-1 <<http://sharif.edu/~stkanchi/ET/>>, accessed at 08.05.2012

Appendix A

SPD25 JACKET

**FABRICATED WEIGHT DATA
FROM BOCAD**

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Elevation At Row 1	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8047	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	1	ø 610.0 x 15.88	EN10025 S355J0	12.820	12.820	232.670	2982.83	2839.25	12.203	3.65
2	1	ø 762.0 x 12.70	EN10025 S355J0	18.437	18.437	234.680	4326.80	4237.74	18.058	8.42
3	1	ø 762.0 x 25.40	EN10025 S355K2 Z35	2.500	2.500	461.410	1153.53	1153.53	2.500	1.17
4	1	ø 762.0 x 12.70	EN10025 S355J0	15.001	15.001	234.680	3520.43	3470.93	14.790	6.90
5	1	ø 762.0 x 12.70	EN10025 S355J0	15.964	15.964	234.680	3746.43	3622.07	15.434	7.20
6	1	ø 762.0 x 12.70	EN10025 S355J0	14.326	14.326	234.680	3362.03	3274.35	13.952	6.51
7	1	ø 762.0 x 12.70	EN10025 S355J0	4.455	4.455	234.680	1045.50	997.65	4.251	1.98
8	1	ø 762.0 x 25.40	EN10025 S355K2 Z35	1.200	1.200	461.410	553.69	553.69	1.200	0.56
9	1	ø 610.0 x 15.88	EN10025 S355J0	17.180	17.180	232.670	3997.27	3860.69	16.593	4.96
10	1	ø 610.0 x 15.88	EN10025 S355J0	4.973	4.973	232.670	1157.07	1107.59	4.760	1.42
11	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.200	1.200	366.190	439.43	439.43	1.200	0.36
12	1	ø 610.0 x 15.88	EN10025 S355J0	6.776	6.776	232.670	1576.57	1529.55	6.574	1.97
13	1	ø 610.0 x 12.70	EN10025 S355J0	12.202	12.202	187.080	2282.75	2210.76	11.817	3.53
14	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.500	1.500	366.190	549.29	549.29	1.500	0.45
15	1	ø 610.0 x 12.70	EN10025 S355J0	16.430	16.430	187.080	3073.72	2959.70	15.821	4.73
16	1	ø 610.0 x 15.88	EN10025 S355J0	11.422	11.422	232.670	2657.56	2557.04	10.990	3.29
17	1	ø 610.0 x 38.10	EN10025 S355K2 Z35	3.150	3.150	537.360	1692.68	1692.68	3.150	0.94
18	1	ø 610.0 x 31.75	EN10025 S355J0	9.207	9.207	452.770	4168.65	4001.73	8.838	2.64
19	1	ø 610.0 x 31.75	EN10025 S355J0	9.957	9.957	452.770	4508.23	4240.89	9.367	2.80
20	1	ø 610.0 x 31.75	EN10025 S355J0	2.890	2.890	452.770	1308.51	1208.40	2.669	0.80
21	1	ø 610.0 x 15.88	EN10025 S355J0	2.516	2.516	232.670	585.40	585.40	2.516	0.75
22	1	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.200	1.200	537.360	644.83	644.83	1.200	0.36
23	1	ø 610.0 x 15.88	EN10025 S355J0	7.209	7.209	232.670	1677.32	1557.29	6.693	2.00

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Elevation At Row 2	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8048	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	1	ø 457.0 x 19.05	API5L X52 SR4 & SR5	1.556	1.556	205.750	320.15	320.15	1.556	0.26
2	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	8.026	8.026	139.160	1116.90	1079.98	7.761	1.30
3	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	2.690	2.690	139.160	374.34	360.78	2.593	0.44
4	1	ø 762.0 x 19.05	EN10025 S355J0	19.970	19.970	349.040	6970.33	6684.76	19.152	8.93
5	1	ø 762.0 x 12.70	EN10025 S355J0	15.847	15.847	234.680	3718.97	3609.95	15.382	7.18
6	1	ø 762.0 x 12.70	EN10025 S355J0	14.948	14.948	234.680	3508.00	3459.27	14.740	6.88
7	1	ø 762.0 x 25.40	EN10025 S355K2 Z35	2.300	2.300	461.410	1061.24	1061.24	2.300	1.07
8	1	ø 762.0 x 19.05	EN10025 S355J0	18.570	18.570	349.040	6481.67	6292.07	18.027	8.41
9	1	ø 610.0 x 15.88	EN10025 S355J0	16.358	16.358	232.670	3806.02	3717.19	15.976	4.78
10	1	ø 610.0 x 15.88	EN10025 S355J0	12.151	12.151	232.670	2827.17	2776.99	11.935	3.57
11	1	ø 610.0 x 15.88	EN10025 S355J0	12.898	12.898	232.670	3000.98	2862.97	12.305	3.68
12	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.500	1.500	366.190	549.29	549.29	1.500	0.45
13	1	ø 610.0 x 15.88	EN10025 S355J0	17.101	17.101	232.670	3978.89	3789.92	16.289	4.87
14	1	ø 610.0 x 15.88	EN10025 S355J0	11.354	11.354	232.670	2641.74	2541.37	10.923	3.27
15	1	ø 610.0 x 31.75	EN10025 S355K2	2.399	2.399	452.770	1086.20	973.85	2.151	0.64
16	1	ø 610.0 x 31.75	EN10025 S355K2	9.910	9.910	452.770	4486.95	4221.46	9.324	2.79
17	1	ø 610.0 x 31.75	EN10025 S355K2	9.163	9.163	452.770	4148.73	4038.78	8.920	2.67
18	1	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.500	1.500	537.360	806.04	806.04	1.500	0.45
19	1	ø 610.0 x 25.40	EN10025 S355J0	1.650	1.650	366.190	604.21	604.21	1.650	0.49
20	1	ø 610.0 x 15.88	EN10025 S355J0	11.351	11.351	232.670	2641.04	2505.73	10.769	3.22

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Elevation At Row A		Date 17.04.09
Drawing No.	SP17/18-P3-ST-DW-8049		Rev 2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	11.658	11.658	139.160	1622.33	1560.66	11.215	1.88
2	1	ø 864.0 x 25.40	EN10025 S355J0	18.516	18.516	525.300	9726.45	9351.69	17.803	10.68
3	1	ø 864.0 x 25.40	EN10025 S355J0	16.577	16.577	525.300	8707.90	8383.15	15.959	9.57
4	1	ø 864.0 x 25.40	EN10025 S355J0	14.690	14.690	525.300	7716.66	7570.93	14.413	8.64
5	1	ø 864.0 x 38.10	EN10025 S355K2 Z35	2.400	2.400	776.020	1862.45	1862.45	2.400	1.44
6	1	ø 864.0 x 38.10	EN10025 S355K2	15.906	15.906	776.020	12343.37	11954.47	15.405	9.24
7	1	ø 610.0 x 15.88	EN10025 S355J0	16.784	16.784	232.670	3900.48	3740.05	16.074	4.81
8	1	ø 610.0 x 15.88	EN10025 S355J0	14.902	14.902	232.670	3467.25	3329.85	14.311	4.28
9	1	ø 660.0 x 38.10	EN10025 S355K2	13.374	13.374	584.340	7814.96	7670.79	13.127	4.59
10	1	ø 660.0 x 38.10	EN10025 S355K2 Z35	1.500	1.500	584.340	876.51	876.51	1.500	0.52
11	1	ø 660.0 x 38.10	EN10025 S355K2	15.124	15.124	584.340	8837.56	8651.84	14.806	5.18
12	1	ø 610.0 x 15.88	EN10025 S355J0	12.095	12.095	232.670	2814.14	2732.38	11.744	3.51
13	1	ø 610.0 x 19.05	EN10025 S355J0	1.673	1.673	277.630	464.47	404.58	1.457	0.44
14	1	ø 610.0 x 25.40	EN10025 S355J0	13.116	13.116	366.190	4802.95	4637.29	12.664	3.79
15	1	ø 660.0 x 38.10	EN10025 S355K2	11.722	11.722	584.340	6849.63	6543.92	11.199	3.92
16	1	ø 660.0 x 31.75	EN10025 S355K2	11.994	11.994	491.920	5900.09	5748.96	11.687	4.09
17	1	ø 914.0 x 38.10	EN10025 S355K2 Z35	1.500	1.500	823.000	1234.50	1234.50	1.500	1.01
18	1	ø 914.0 x 25.40	EN10025 S355J0	7.915	7.915	556.820	4405.65	4405.65	7.915	5.31
19	1	ø 914.0 x 50.80	EN10025 S355NL Z35	2.500	2.500	1081.420	2703.55	2703.55	2.500	1.68
20	1	ø 914.0 x 38.10	API5L X52 SR4 & SR5	7.025	7.025	823.000	5781.58	5781.58	7.025	4.72
21	1	ø 914.0 x 38.10	EN10025 S355K2 Z35	1.160	1.160	823.000	954.68	954.68	1.160	0.78
22	2	PL25mm	EN10025 S355J0	0.606	1.212	196.250	237.86	237.86	1.212	0.03
23	1	ø 660.0 x 38.10	EN10025 S355K2	1.689	1.689	584.340	986.95	847.95	1.451	0.51

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Elevation At Row B		Date 17.04.09
Drawing No.	SP17/18-P3-ST-DW-8050		Rev 2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	11.661	11.661	139.160	1622.74	1561.17	11.219	1.88
2	1	ø 864.0 x 25.40	EN10025 S355J0	18.516	18.516	525.300	9726.45	9353.17	17.805	10.68
3	1	ø 864.0 x 25.40	EN10025 S355J0	16.578	16.578	525.300	8708.42	8385.15	15.963	9.57
4	1	ø 864.0 x 25.40	EN10025 S355J0	14.690	14.690	525.300	7716.66	7570.93	14.413	8.64
5	1	ø 864.0 x 38.10	EN10025 S355K2 Z35	2.400	2.400	776.020	1862.45	1862.45	2.400	1.44
6	1	ø 864.0 x 38.10	EN10025 S355K2	15.906	15.906	776.020	12343.37	11954.47	15.405	9.24
7	1	ø 610.0 x 15.88	EN10025 S355J0	16.138	16.138	232.670	3754.83	3676.08	15.800	4.72
8	1	ø 610.0 x 15.88	EN10025 S355J0	14.282	14.282	232.670	3322.99	3261.44	14.017	4.19
9	1	ø 610.0 x 25.40	EN10025 S355J0	14.118	14.118	366.190	5169.87	4938.62	13.486	4.03
10	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.500	3.000	537.360	1612.08	1612.08	3.000	0.90
11	1	ø 610.0 x 25.40	EN10025 S355J0	15.874	15.874	366.190	5812.90	5548.92	15.153	4.53
12	1	ø 610.0 x 15.88	EN10025 S355J0	14.617	14.617	232.670	3400.94	3232.55	13.893	4.15
13	1	ø 610.0 x 25.40	EN10025 S355J0	13.072	13.072	366.190	4786.84	4605.53	12.577	3.76
14	1	ø 610.0 x 25.40	EN10025 S355J0	12.994	12.994	366.190	4758.27	4643.80	12.681	3.79
15	1	ø 610.0 x 38.10	EN10025 S355K2	11.543	11.543	537.360	6202.75	6069.66	11.295	3.38

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Plan at EL (+)5.750m L.A.T		Date 17.04.09
Drawing No.	SP17/18-P3-ST-DW-8051		Rev 2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	2.100	2.100	366.190	769.00	769.00	2.100	0.63
2	1	ø 610.0 x 12.70	EN10025 S355J0	3.242	3.242	187.080	606.51	606.51	3.242	0.97
3	1	ø 610.0 x 31.75	EN10025 S355K2 Z35	9.500	9.500	452.770	4301.32	4301.32	9.500	2.84
4	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	2.127	2.127	366.190	778.89	751.83	2.053	0.61
5	1	ø 610.0 x 31.75	EN10025 S355K2 Z35	9.050	9.050	452.770	4097.57	4097.57	9.050	2.71
6	1	ø 610.0 x 25.40	EN10025 S355J0	2.697	2.697	366.190	987.61	896.24	2.447	0.73
7	2	ø 610.0 x 25.40	EN10025 S355J0	1.668	3.336	366.190	1221.61	1173.44	3.204	0.96
8	1	ø 610.0 x 25.40	EN10025 S355J0	2.359	2.359	366.190	863.84	824.90	2.253	0.67
9	2	ø 355.6 x 15.88	API5L X52 SR4 & SR5	9.238	18.476	133.040	2458.05	2387.65	17.947	1.82
10	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	2.800	2.800	366.190	1025.33	1025.33	2.800	0.84
11	1	ø 610.0 x 15.88	EN10025 S355J0	5.686	5.686	232.670	1322.96	1322.96	5.686	1.70
12	2	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.800	3.600	366.190	1318.28	1318.28	3.600	1.08
13	2	ø 610.0 x 25.40	EN10025 S355J0	1.626	3.252	366.190	1190.85	1150.49	3.142	0.94
14	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.011	1.011	366.190	370.22	341.10	0.931	0.28
15	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.211	1.211	366.190	443.46	413.14	1.128	0.34
16	1	ø 610.0 x 15.88	EN10025 S355J0	6.765	6.765	232.670	1574.01	1574.01	6.765	2.02
17	2	ø 355.6 x 19.05	API5L X52 SR4 & SR5	1.468	2.936	158.110	464.21	442.98	2.802	0.28
18	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	10.867	10.867	139.160	1512.25	1457.05	10.470	1.76
19	1	ø 323.9 x 12.70	API5L X52 SR4 & SR5	5.122	5.122	97.470	499.24	469.43	4.816	0.41
20	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.524	9.048	97.470	881.91	833.50	8.551	0.72
21	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.251	8.502	97.470	828.69	772.83	7.929	0.67
22	2	ø 355.6 x 15.88	API5L X52 SR4 & SR5	10.705	21.410	133.040	2848.39	2760.18	20.747	2.11
23	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.220	13.320	97.470	1298.30	1194.27	12.253	1.03
24	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.297	13.782	97.470	1343.33	1238.04	12.702	1.07
25	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.374	14.244	97.470	1388.36	1281.96	13.152	1.11
26	1	ø 457.0 x 12.70	API5L X52 SR4 & SR5	10.899	10.899	139.160	1516.70	1461.56	10.503	1.76
27	1	ø 323.9 x 12.70	API5L X52 SR4 & SR5	5.092	5.092	97.470	496.32	466.58	4.787	0.40
28	1	ø 610.0 x 15.88	EN10025 S355J0	5.618	5.618	232.670	1307.14	1307.14	5.618	1.68
29	1	ø 610.0 x 25.40	EN10025 S355J0	1.809	1.809	366.190	662.44	637.29	1.740	0.52
30	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.220	1.220	366.190	446.75	446.75	1.220	0.36
31	1	ø 610.0 x 25.40	EN10025 S355J0	2.340	2.340	366.190	856.88	856.88	2.340	0.70
32	1	ø 610.0 x 25.40	EN10025 S355J0	6.815	6.815	366.190	2495.58	2441.34	6.667	1.99

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Plan at EL (-)13.000m L.A.T	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8052	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	4	PL25mm	EN10025 S355J0	0.186	0.744	196.250	146.01	146.01	0.744	0.02
2	4	ø 219.1 x 19.05	API5L GRB SR4 & SR5	0.150	0.600	93.980	56.39	56.39	0.600	0.02
3	1	ø 610.0 x 12.70	EN10025 S355J0	16.069	16.069	187.080	3006.19	2992.53	15.996	4.78
4	1	ø 610.0 x 19.05	EN10025 S355J0	2.610	2.610	277.630	724.61	641.56	2.311	0.69
5	1	ø 610.0 x 12.70	EN10025 S355J0	9.909	9.909	187.080	1853.78	1816.72	9.711	2.90
6	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.500	1.500	366.190	549.29	549.29	1.500	0.45
7	1	ø 610.0 x 12.70	EN10025 S355J0	2.663	2.663	187.080	498.19	498.19	2.663	0.80
8	2	ø 610.0 x 19.05	EN10025 S355J0	2.100	4.200	277.630	1166.05	1166.05	4.200	1.26
9	2	ø 610.0 x 12.70	EN10025 S355J0	4.881	9.762	187.080	1826.27	1786.06	9.547	2.85
10	2	ø 610.0 x 25.40	EN10025 S355J0	1.489	2.978	366.190	1090.51	1044.50	2.852	0.85
11	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.500	3.000	537.360	1612.08	1612.08	3.000	0.90
12	2	ø 610.0 x 19.05	EN10025 S355J0	1.358	2.716	277.630	754.04	754.04	2.716	0.81
13	1	ø 610.0 x 31.75	EN10025 S355K2 Z35	9.500	9.500	452.770	4301.32	4301.32	9.500	2.84
14	1	ø 610.0 x 12.70	EN10025 S355J0	3.074	3.074	187.080	575.08	554.30	2.963	0.89
15	1	ø 610.0 x 12.70	EN10025 S355J0	2.127	2.127	366.190	778.89	389.84	1.065	0.32
16	1	ø 610.0 x 12.70	EN10025 S355J0	10.322	10.322	187.080	1931.04	1911.32	10.217	3.05
18	1	ø 457.0 x 15.88	API5L X52 SR4 & SR5	10.587	10.587	172.750	1828.90	1788.57	10.354	1.74
19	1	ø 457.0 x 19.05	API5L X52 SR4 & SR5	1.300	1.300	205.750	267.48	267.48	1.300	0.22
20	1	ø 457.0 x 15.88	API5L X52 SR4 & SR5	7.143	7.143	172.750	1233.95	1190.45	6.891	1.16
21	1	ø 457.0 x 15.88	API5L X52 SR4 & SR5	11.237	11.237	172.750	1941.19	1870.62	10.829	1.82
22	1	ø 457.0 x 15.88	API5L X52 SR4 & SR5	7.793	7.793	172.750	1346.24	1276.10	7.387	1.24
23	1	ø 610.0 x 19.05	EN10025 S355J0	2.272	2.272	277.630	630.78	586.95	2.114	0.63
24	1	ø 610.0 x 31.75	EN10025 S355K2 Z35	12.851	12.851	452.770	5818.55	5672.69	12.529	3.75
25	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	4.623	9.246	107.400	993.02	911.10	8.483	0.86
26	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	4.099	8.198	107.400	880.47	813.98	7.579	0.77
27	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	3.486	6.972	107.400	748.79	713.03	6.639	0.67
28	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	1.083	2.166	302.300	654.78	623.46	2.062	0.43
29	2	ø 508.0 x 15.88	API5L X52 SR4 & SR5	5.327	10.654	192.730	2053.35	2053.35	10.654	2.21
30	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	1.950	3.900	302.300	1178.97	1178.97	3.900	0.81
31	2	ø 508.0 x 25.40	EN10025 S355J0	2.209	4.418	302.300	1335.56	1264.92	4.184	0.87
32	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	4.864	9.728	107.400	1044.79	963.75	8.973	0.91
33	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	4.365	8.730	107.400	937.60	868.37	8.085	0.82
34	2	ø 355.6 x 12.70	API5L X52 SR4 & SR5	4.044	8.088	107.400	868.65	805.08	7.496	0.76
35	4	ø 355.6 x 15.88	API5L X52 SR4 & SR5	10.705	42.820	133.040	5696.77	5520.35	41.494	4.22
36	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.220	13.320	97.470	1298.30	1194.14	12.251	1.03
37	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.297	13.782	97.470	1343.33	1238.04	12.702	1.07
38	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.374	14.244	97.470	1388.36	1281.96	13.152	1.11

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Plan at EL (-)35.000m L.A.T	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8053	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	4	PL25mm	EN10025 S355J0	0.186	0.744	196.250	146.01	146.01	0.744	0.02
2	1	ø 610.0 x 12.70	EN10025 S355J0	7.631	7.631	187.080	1427.61	1397.08	7.466	2.23
3	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	2.000	2.000	366.190	732.38	732.38	2.000	0.60
4	1	ø 610.0 x 12.70	EN10025 S355J0	9.114	9.114	187.080	1705.05	1705.05	9.114	2.72
5	2	ø 660.0 x 12.70	EN10025 S355J0	12.667	25.334	202.740	5136.22	5036.15	24.840	8.69
6	4	ø 219.1 x 19.05	API5L GRB SR4 & SR5	0.150	0.600	93.980	56.39	56.39	0.600	0.02
7	1	ø 610.0 x 12.70	EN10025 S355J0	2.390	2.390	187.080	447.12	447.12	2.390	0.71
8	2	ø 660.0 x 25.40	EN10025 S355K2 Z35	1.500	3.000	397.510	1192.53	1192.53	3.000	1.05
9	2	ø 660.0 x 12.70	EN10025 S355J0	9.652	19.304	202.740	3913.69	3837.06	18.926	6.62
10	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.220	1.220	366.190	446.75	446.75	1.220	0.36
11	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.006	8.012	97.470	780.93	711.07	7.295	0.61
12	2	ø 610.0 x 15.88	EN10025 S355J0	4.632	9.264	232.670	2155.45	2102.65	9.037	2.70
13	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.500	3.000	537.360	1612.08	1612.08	3.000	0.90
14	2	ø 610.0 x 19.05	EN10025 S355J0	1.358	2.716	277.630	754.04	754.04	2.716	0.81
15	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	9.500	9.500	366.190	3478.81	3478.81	9.500	2.84
16	2	ø 508.0 x 31.75	EN10025 S355K2	14.402	28.804	372.910	10741.30	10419.56	27.941	5.79
17	2	ø 508.0 x 12.70	EN10025 S355J0	15.135	30.270	155.130	4695.79	4623.14	29.802	6.18
18	2	ø 610.0 x 12.70	EN10025 S355J0	5.634	11.268	187.080	2108.02	2008.41	10.736	3.21
19	2	ø 610.0 x 25.40	EN10025 S355K2 Z35	0.710	1.420	366.190	519.99	471.27	1.287	0.38
20	2	ø 610.0 x 19.05	EN10025 S355J0	1.555	3.110	277.630	863.43	863.43	3.110	0.93
21	1	ø 610.0 x 25.40	EN10025 S355K2 Z35	8.320	8.320	366.190	3046.70	3046.70	8.320	2.49
22	2	ø 508.0 x 12.70	API5L X52 SR4 & SR5	11.599	23.198	155.130	3598.71	3375.31	21.758	4.51
23	2	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.557	3.114	366.190	1140.32	1081.43	2.953	0.88
24	2	ø 610.0 x 25.40	EN10025 S355J0	4.945	9.890	366.190	3621.62	3621.62	9.890	2.96
25	2	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.850	3.700	366.190	1354.90	1354.90	3.700	1.11
26	2	ø 610.0 x 25.40	EN10025 S355J0	2.415	4.830	366.190	1768.70	1589.31	4.340	1.30
27	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.793	9.586	97.470	934.35	850.57	8.726	0.74
28	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.268	8.536	97.470	832.00	760.90	7.806	0.66
29	4	ø 355.6 x 15.88	API5L X52 SR4 & SR5	10.705	42.820	133.040	5696.77	5520.35	41.494	4.22
30	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.221	13.326	97.470	1298.89	1194.81	12.258	1.03
31	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.297	13.782	97.470	1343.33	1238.04	12.702	1.07
32	6	ø 323.9 x 12.70	API5L X52 SR4 & SR5	2.374	14.244	97.470	1388.36	1281.96	13.152	1.11
33	1	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.127	2.127	537.360	1142.96	1102.70	2.052	0.61

WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD 25 - Jacket Plan at EL (-)58.500m L.A.T	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8054	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	2	ø 660.0 x 12.70	EN10025 S355J0	8.174	16.348	202.740	3314.39	3242.79	15.995	5.60
2	2	ø 660.0 x 19.05	EN10025 S355J0	1.150	2.300	301.120	692.58	692.58	2.300	0.80
3	2	ø 660.0 x 12.70	EN10025 S355J0	5.283	10.566	202.740	2142.15	2142.15	10.566	3.70
5	1	ø 660.0 x 31.75	EN10025 S355K2 Z35	2.000	2.000	491.920	983.84	983.84	2.000	0.70
6	1	ø 660.0 x 15.88	EN10025 S355J0	7.217	7.217	252.250	1820.49	1780.72	7.059	2.47
7	2	ø 660.0 x 19.05	EN10025 S355J0	1.100	2.200	301.120	662.46	662.46	2.200	0.77
8	2	ø 660.0 x 12.70	EN10025 S355J0	5.437	10.874	202.740	2204.59	2204.59	10.874	3.81
9	2	ø 660.0 x 25.40	EN10025 S355K2 Z35	3.350	6.700	397.510	2663.32	2663.32	6.700	2.34
10	2	ø 660.0 x 12.70	EN10025 S355J0	9.652	19.304	202.740	3913.69	3837.06	18.926	6.62
11	2	ø 660.0 x 12.70	EN10025 S355J0	7.997	15.994	202.740	3242.62	3172.73	15.649	5.48
12	2	ø 660.0 x 31.75	EN10025 S355K2 Z35	1.500	3.000	491.920	1475.76	1475.76	3.000	1.05
13	2	ø 660.0 x 19.05	EN10025 S355J0	1.358	2.716	301.120	817.84	817.84	2.716	0.95
14	1	ø 660.0 x 25.40	EN10025 S355K2 Z35	9.500	9.500	397.510	3776.35	3776.35	9.500	3.32
15	1	ø 660.0 x 12.70	EN10025 S355J0	7.217	7.217	202.740	1463.17	1431.20	7.059	2.47
16	1	ø 508.0 x 15.88	API5L X52 SR4 & SR5	4.700	4.700	192.730	905.83	877.77	4.554	0.94
17	1	ø 508.0 x 12.70	API5L X52 SR4 & SR5	2.107	2.107	155.130	326.86	326.86	2.107	0.44
18	2	ø 508.0 x 31.75	EN10025 S355K2 Z35	3.400	6.800	372.910	2535.79	2535.79	6.800	1.41
19	2	ø 508.0 x 12.70	API5L X52 SR4 & SR5	6.270	12.540	155.130	1945.33	1903.32	12.269	2.54
20	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	1.233	2.466	302.300	745.47	712.67	2.358	0.49
21	2	ø 508.0 x 15.88	API5L X52 SR4 & SR5	4.390	8.780	192.730	1692.17	1692.17	8.780	1.82
22	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	3.500	7.000	302.300	2116.10	2116.10	7.000	1.45
23	2	ø 508.0 x 15.88	API5L X52 SR4 & SR5	1.403	2.806	192.730	540.80	506.15	2.626	0.54
24	1	ø 508.0 x 12.70	API5L X52 SR4 & SR5	6.805	6.805	155.130	1055.66	1028.10	6.627	1.37
25	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.793	9.586	97.470	934.35	873.76	8.964	0.76
26	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.344	8.688	97.470	846.82	794.69	8.153	0.69
27	2	ø 323.9 x 12.70	API5L X52 SR4 & SR5	4.027	8.054	97.470	785.02	732.19	7.512	0.63
28	2	ø 355.6 x 15.88	API5L X52 SR4 & SR5	10.677	21.354	133.040	2840.94	2753.17	20.694	2.10
29	2	ø 457.0 x 12.70	API5L X52 SR4 & SR5	9.206	18.412	139.160	2562.21	2481.20	17.830	2.99
30	1	ø 508.0 x 19.05	API5L X52 SR4 & SR5	13.716	13.716	229.710	3150.70	3020.95	13.151	2.73
31	2	ø 610.0 x 12.70	EN10025 S355J0	8.991	17.982	187.080	3364.07	3243.95	17.340	5.18
32	2	ø 610.0 x 12.70	EN10025 S355J0	2.264	4.528	187.080	847.10	813.41	4.348	1.30
33	2	ø 610.0 x 25.40	EN10025 S355K2 Z35	1.400	2.800	366.190	1025.33	1025.33	2.800	0.84
34	1	ø 610.0 x 12.70	EN10025 S355J0	5.522	5.522	187.080	1033.06	1033.06	5.522	1.65
35	2	ø 508.0 x 12.70	API5L X52 SR4 & SR5	4.655	9.310	155.130	1444.26	1424.24	9.181	1.90
36	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	2.500	5.000	302.300	1511.50	1511.50	5.000	1.04
37	2	ø 508.0 x 12.70	API5L X52 SR4 & SR5	4.556	9.112	155.130	1413.54	1351.86	8.714	1.81
38	2	ø 508.0 x 12.70	API5L X52 SR4 & SR5	12.062	24.124	155.130	3742.36	3623.91	23.360	4.84
39	2	ø 457.0 x 12.70	API5L X52 SR4 & SR5	10.276	20.552	139.160	2860.02	2760.13	19.834	3.33
40	2	ø 508.0 x 12.70	EN10025 S355J0	4.222	8.444	155.130	1309.92	1237.30	7.976	1.65
41	2	ø 508.0 x 12.70	EN10025 S355J0	6.172	12.344	155.130	1914.92	1900.36	12.250	2.54
43	4	PL25mm	EN10025 S355J0	0.242	0.968	196.250	189.97	189.97	0.968	0.02
44	4	ø 219.0 x 19.05	API5L GRB SR4 & SR5	0.150	0.600	93.940	56.36	56.36	0.600	0.02
45	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	3.000	6.000	302.300	1813.80	1813.80	6.000	1.24

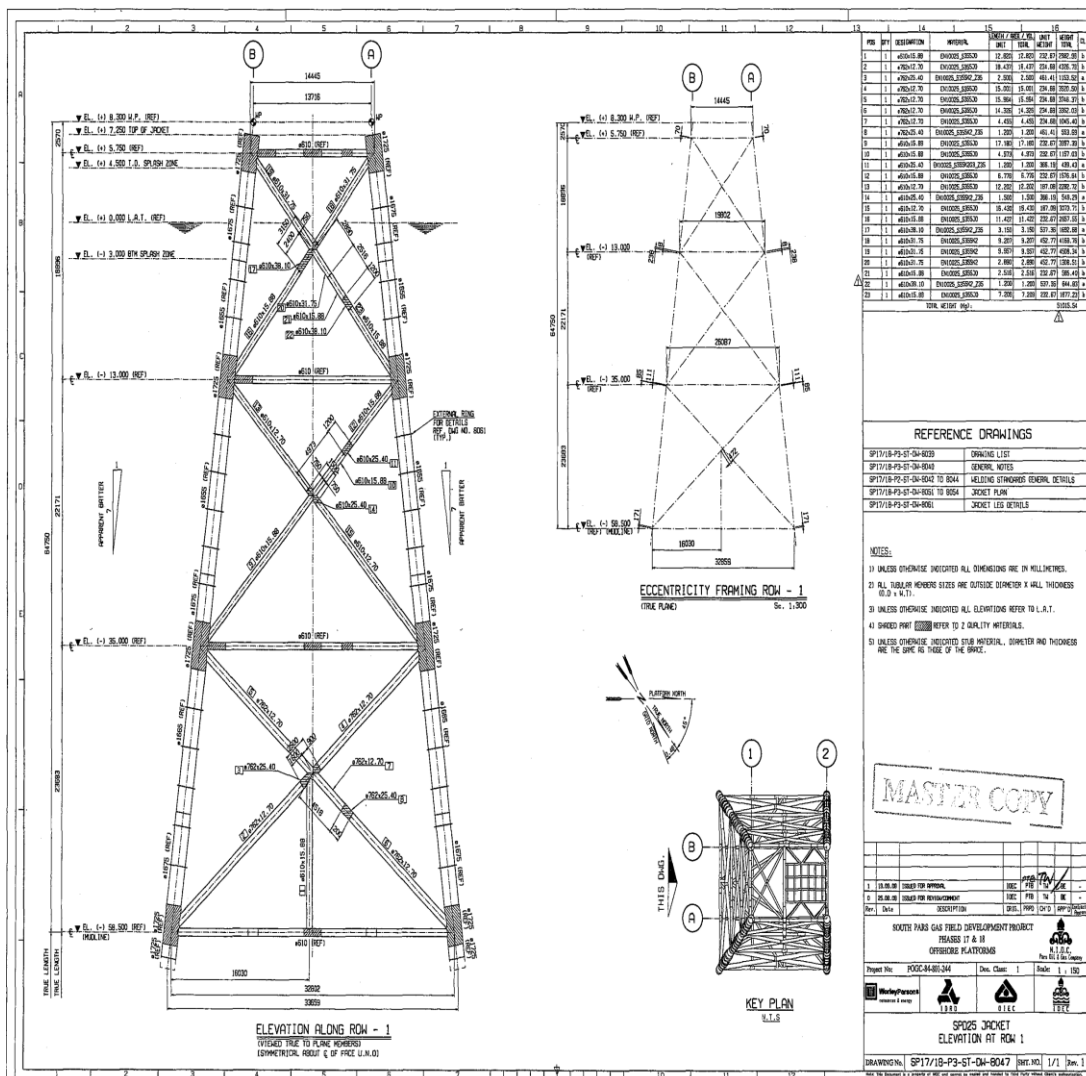
WORLEYPARSONS			
SOUTH PARS GAS FIELD DEVELOPMENT PROJECT PHASE 17 & 18			
FABRICATED WEIGHT DATA FROM BOCAD			
Description	SPD25 - Jacket Launch Truss Row A & Row B	Date	17.04.09
Drawing No.	SP17/18-P3-ST-DW-8093	Rev	2

Mark	Qty	Profile	Material Grade	Length/Area m/m ²	Total Length/ Area m/m ²	Unit Weight	Gross Weight (kg)	Net Weight (kg)	Adjusted Total Length	Buoyancy (mt)
1	4	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.150	8.600	537.360	4621.30	4621.30	8.600	2.57
2	2	ø 610.0 x 25.40	EN10025 S355J0	4.575	9.150	366.190	3350.64	3350.64	9.150	2.74
3	4	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.400	5.600	537.360	3009.22	3009.22	5.600	1.67
4	2	ø 610.0 x 25.40	EN10025 S355J0	4.625	9.250	366.190	3387.26	3387.26	9.250	2.77
5	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.250	4.500	537.360	2418.12	2418.12	4.500	1.35
6	2	ø 610.0 x 25.40	EN10025 S355J0	3.000	6.000	366.190	2197.14	2197.14	6.000	1.79
7	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	1.500	3.000	537.360	1612.08	1612.08	3.000	0.90
8	2	ø 610.0 x 25.40	EN10025 S355J0	1.930	3.860	366.190	1413.49	1413.49	3.860	1.15
9	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.640	5.280	537.360	2837.26	2837.26	5.280	1.58
10	2	ø 610.0 x 25.40	EN10025 S355J0	1.480	2.960	366.190	1083.92	1083.92	2.960	0.88
11	2	ø 610.0 x 25.40	EN10025 S355J0	1.850	3.700	366.190	1354.90	1354.90	3.700	1.11
12	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.900	5.800	537.360	3116.69	3116.69	5.800	1.73
13	2	ø 610.0 x 25.40	EN10025 S355J0	3.350	6.700	366.190	2453.47	2453.47	6.700	2.00
14	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.400	4.800	537.360	2579.33	2579.33	4.800	1.44
15	2	ø 610.0 x 25.40	EN10025 S355J0	7.400	14.800	366.190	5419.61	5419.61	14.800	4.42
16	4	ø 610.0 x 38.10	EN10025 S355K2	1.500	6.000	537.360	3224.16	3224.16	4.215	1.26
17	2	ø 610.0 x 38.10	EN10025 S355K2	1.380	2.760	537.360	1483.11	1092.68	2.033	0.61
18	2	ø 508.0 x 19.05	EN10025 S355J0	13.738	27.476	229.710	6311.51	6163.97	26.834	5.56
19	2	ø 508.0 x 25.40	EN10025 S355J0	12.535	25.070	302.300	7578.66	7301.39	24.153	5.01
20	2	ø 508.0 x 31.75	EN10025 S355J0	13.730	27.460	372.910	10240.11	9978.41	26.758	5.55
21	2	ø 610.0 x 19.05	EN10025 S355J0	7.143	14.286	277.630	3966.22	3804.50	13.704	4.10
22	2	ø 610.0 x 38.10	EN10025 S355K2 Z35	2.800	5.600	537.360	3009.22	3009.22	5.600	1.67
23	2	ø 610.0 x 25.40	EN10025 S355J0	4.762	9.524	366.190	3487.59	3371.97	9.208	2.75
24	2	ø 610.0 x 25.40	EN10025 S355J0	7.219	14.438	366.190	5287.05	4785.33	13.068	3.91
25	2	ø 610.0 x 25.40	EN10025 S355J0	8.390	16.780	366.190	6144.67	5801.69	15.843	4.74
26	2	ø 508.0 x 19.05	EN10025 S355J0	8.193	16.386	229.710	3764.03	3611.64	15.723	3.26
27	2	ø 508.0 x 25.40	EN10025 S355J0	7.176	14.352	302.300	4338.61	4012.39	13.273	2.75
28	2	ø 508.0 x 15.88	EN10025 S355J0	4.501	9.002	192.730	1734.96	1662.67	8.627	1.79
29	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	2.600	5.200	302.300	1571.96	1571.96	5.200	1.08
30	2	ø 508.0 x 12.70	EN10025 S355J0	6.988	13.976	155.130	2168.10	2126.45	13.708	2.84
31	2	ø 508.0 x 12.70	EN10025 S355J0	13.605	27.210	155.130	4221.09	4129.12	26.617	5.52
32	2	ø 508.0 x 15.88	EN10025 S355J0	12.533	25.066	192.730	4830.97	4657.26	24.165	5.01
33	2	ø 508.0 x 12.70	EN10025 S355J0	15.145	30.290	155.130	4698.89	4573.79	29.484	6.11
34	4	ø 1067.0 x 31.75	EN10025 S355K2 Z35	1.300	5.200	810.600	4215.12	4215.12	5.200	4.76
35	8	ø 1067x508x20	EN10025 S355J0	1.200	9.600	478.340	3826.72	3634.08	7.597	4.68
36	2	ø 508.0 x 12.70	EN10025 S355J0	6.300	12.600	155.130	1954.64	1954.64	12.600	2.61
37	2	ø 508.0 x 25.40	EN10025 S355K2 Z35	2.650	5.300	302.300	1602.19	1602.19	5.300	1.10
38	2	ø 508.0 x 15.88	EN10025 S355J0	10.000	20.000	192.730	3854.60	3854.60	20.000	4.15
39	2	ø 1067.0 x 31.75	EN10025 S355K2 Z35	3.000	6.000	810.600	4863.60	4863.60	6.000	5.49
40	2	ø 508.0 x 19.05	EN10025 S355J0	11.325	22.650	229.710	5202.93	5202.93	22.650	4.70
41	2	ø 508.0 x 31.75	EN10025 S355K2 Z35	2.600	5.200	372.910	1939.13	1939.13	5.200	1.08
42	2	ø 508.0 x 15.88	EN10025 S355J0	3.525	7.050	192.730	1358.75	1358.75	7.050	1.46
43	2	ø 508.0 x 15.88	EN10025 S355J0	12.154	24.308	192.730	4684.88	4421.38	22.941	4.76

Appendix B

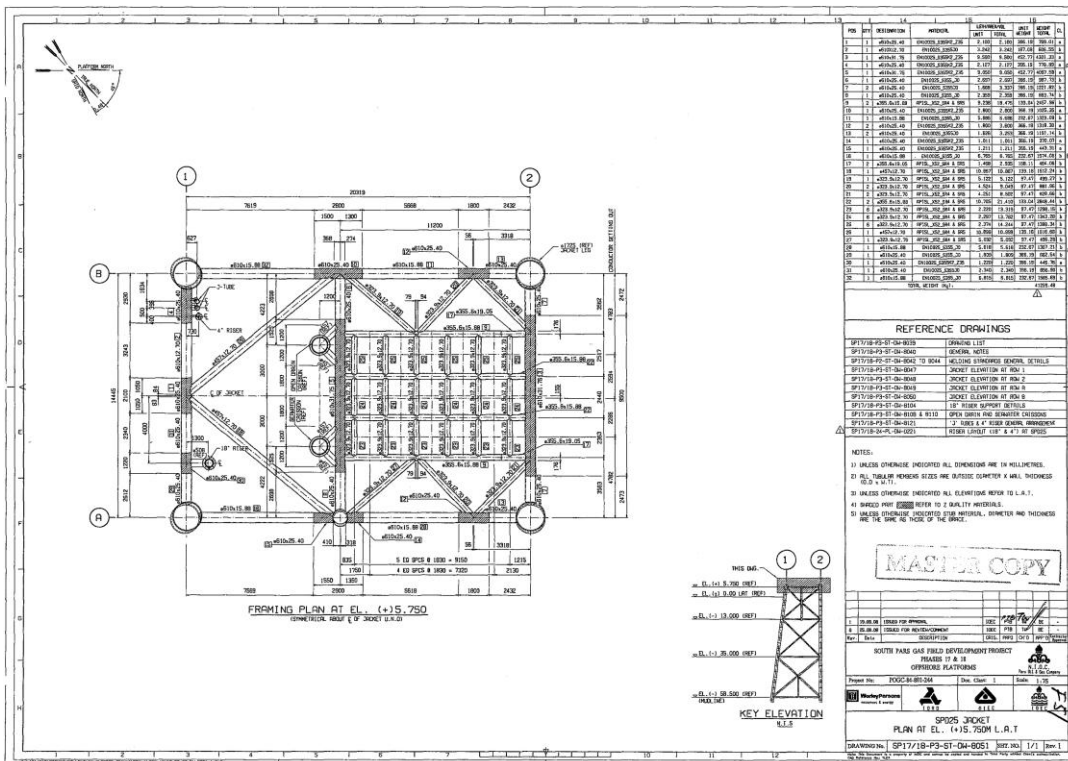
SPD25 JACKET

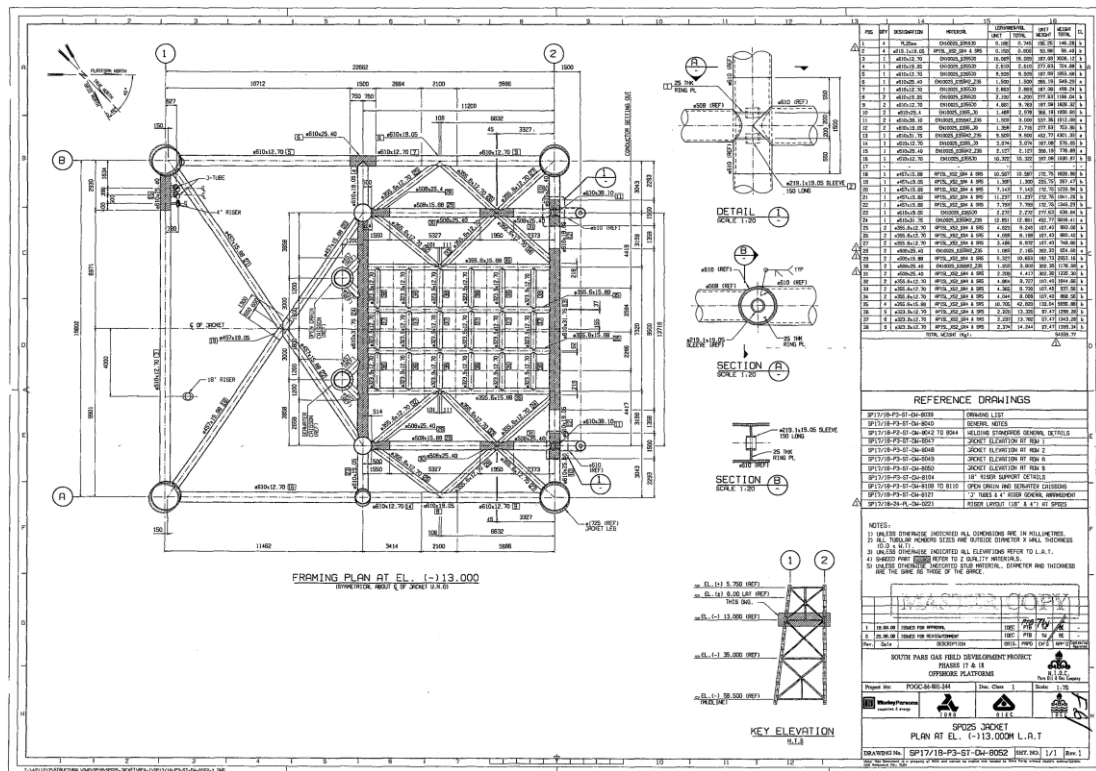
DRAWINGS





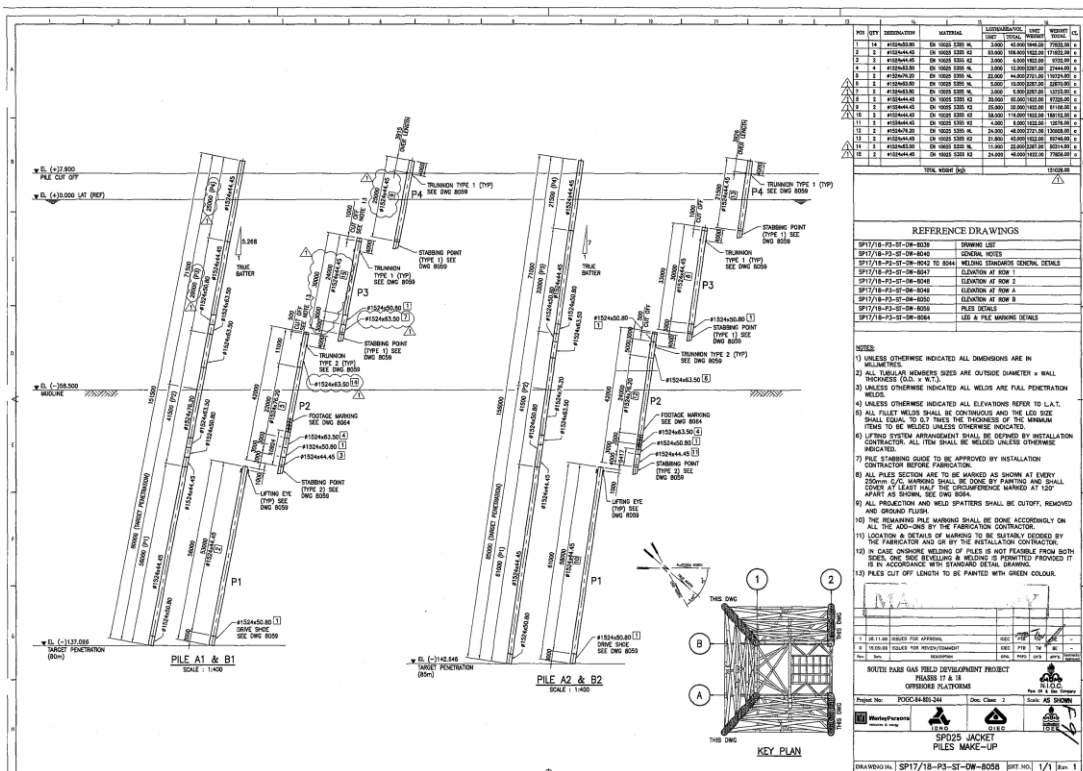


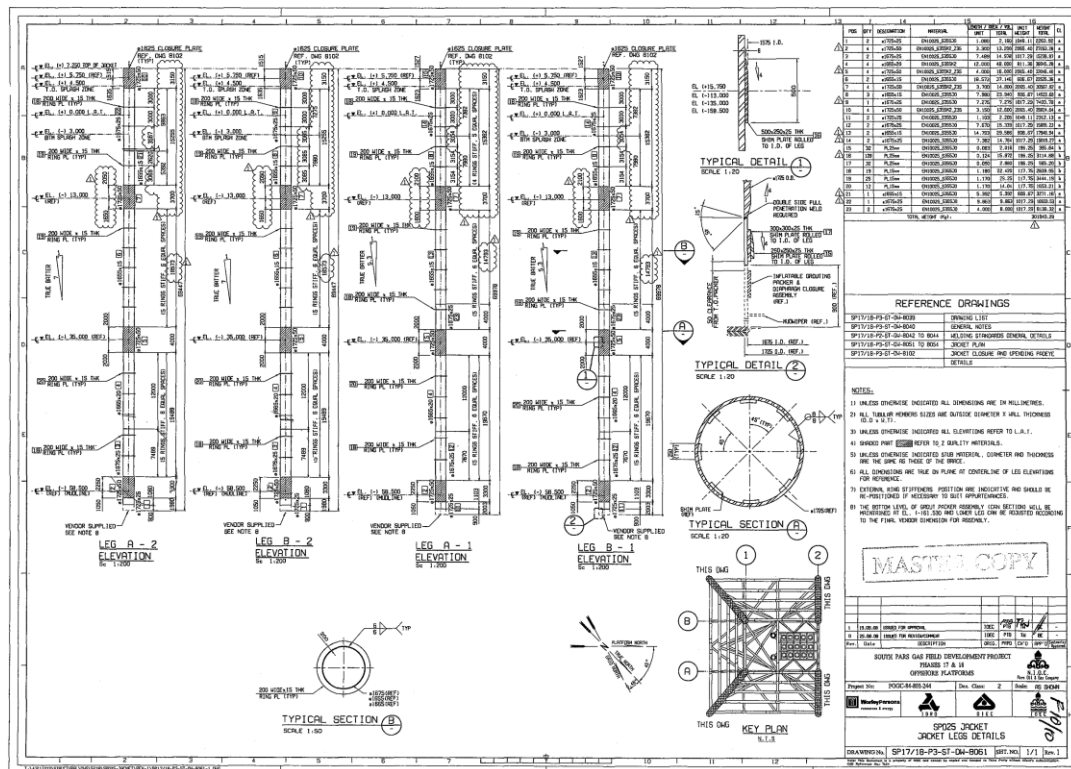












Appendix C

SPD25 JACKET

MISCELLANCE

CALCULATIONS

Equivalent Cross Section for Conductor

	y	A	$M = A.y$	$M_y = A.y^2$	I_g
CONDUCTOR	317.5	50670.7	16087960	5107927284	2558050383
C1	231.02	16039.9	3705537.7	856053319	428289478
C2	165.02	10005.9	1651173.62	272476670.4	136363303
C3	132.11	7379.1	974852.901	128787816.8	64526912.4
TUB	92.685	4851	449614.935	41672560.25	20922069.3
G1	270.68	116673	31581007	8548346985	4319620640
G2	197.675	69118.9	13663076.6	2700848663	1377181406
G3	148.375	22048.1	3271389.81	485392462.3	244237834
G4	112.255	21730.8	2439390.95	273833831.5	139430572
SUM		318517	73824003.5	18415339593	9288622597

$$S = \frac{I_n}{C}, \quad I_n = \sum Ay^2 + \sum I_g - \frac{\sum M^2}{\sum A}$$

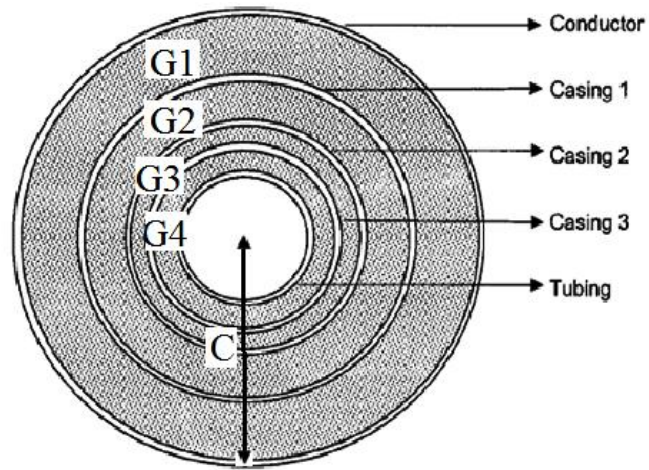
Where:

‘ A ’ is Area, ‘ I_g ’ is element inertia moment, ‘ y ’ is Distance up to neutral axis,

‘ I_n ’ is inertia moment of whole section, ‘ S ’ is Section modulus

$$I_n = 1.059348345E10 \text{ mm}^4, \quad S = \frac{1.059348345E10}{330.2} = 32082021.35 \text{ mm}^3,$$

$$S_{eq} = 33760000 \text{ mm}^3 \Rightarrow \phi 1000 \times 50 \text{ mm}$$



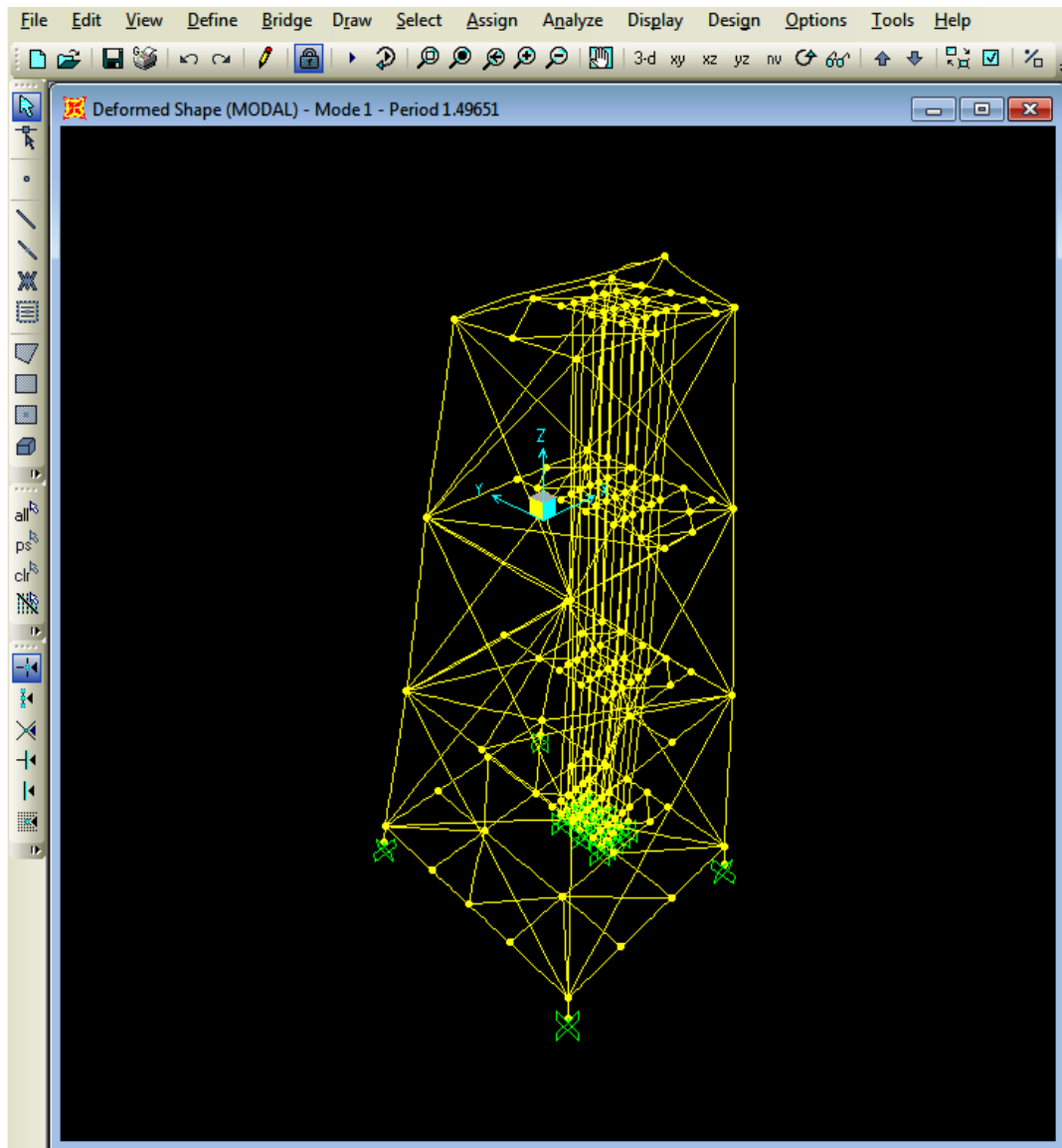
Equivalent Cross Section for Pile & Leg

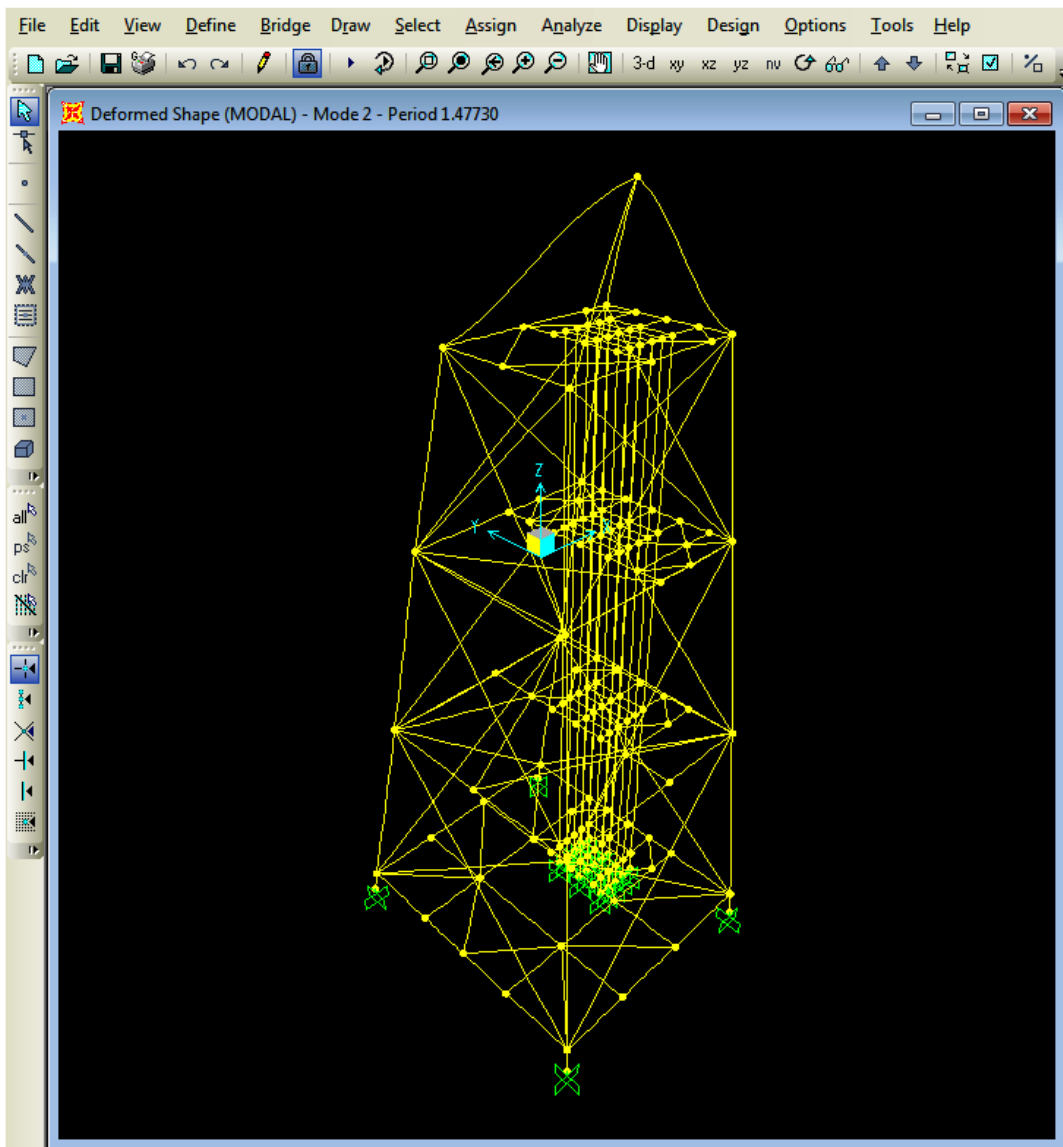
	y	A	$M = A.y$	$M_y = A.y^2$	I_g
LEG	827.5	103987	86048994.3	71205542742	3.56E+10
PILE	739.78	206610	152845924	1.13072E+11	5.66E+10
GROUT	789.75	1720160	1358496534	1.07287E+12	8.60E+10
SUM		2030757	1597391452	1.25715E+12	

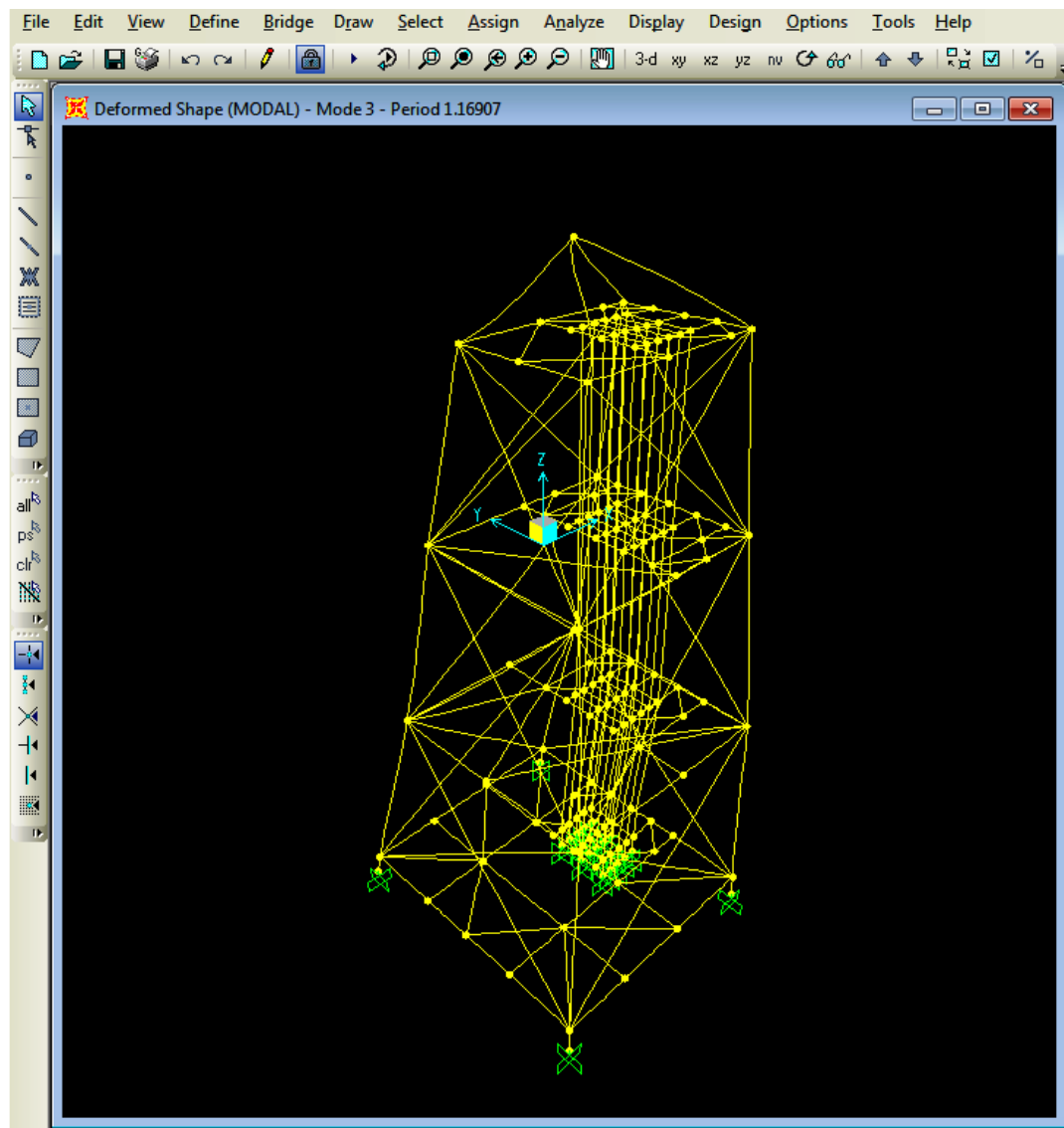
$$I_n = 1.78896E11 \text{ mm}^4, S = \frac{1.78896E11}{837.5} = 213492537.3 \text{ mm}^3$$

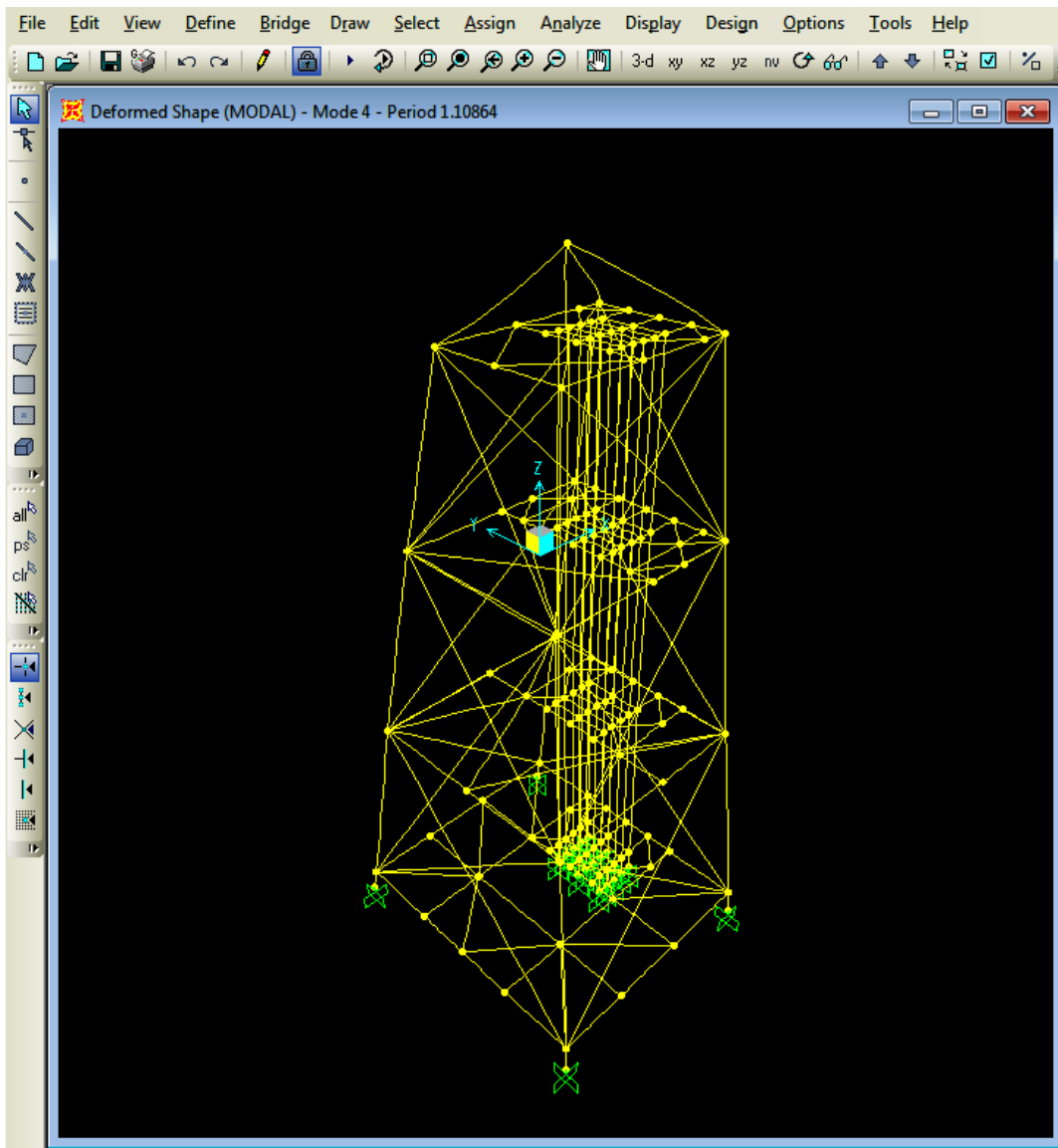
$$S_{eq} = 215559941.4 \text{ mm}^3 \Rightarrow \phi 1800 \times 100 \text{ mm}$$

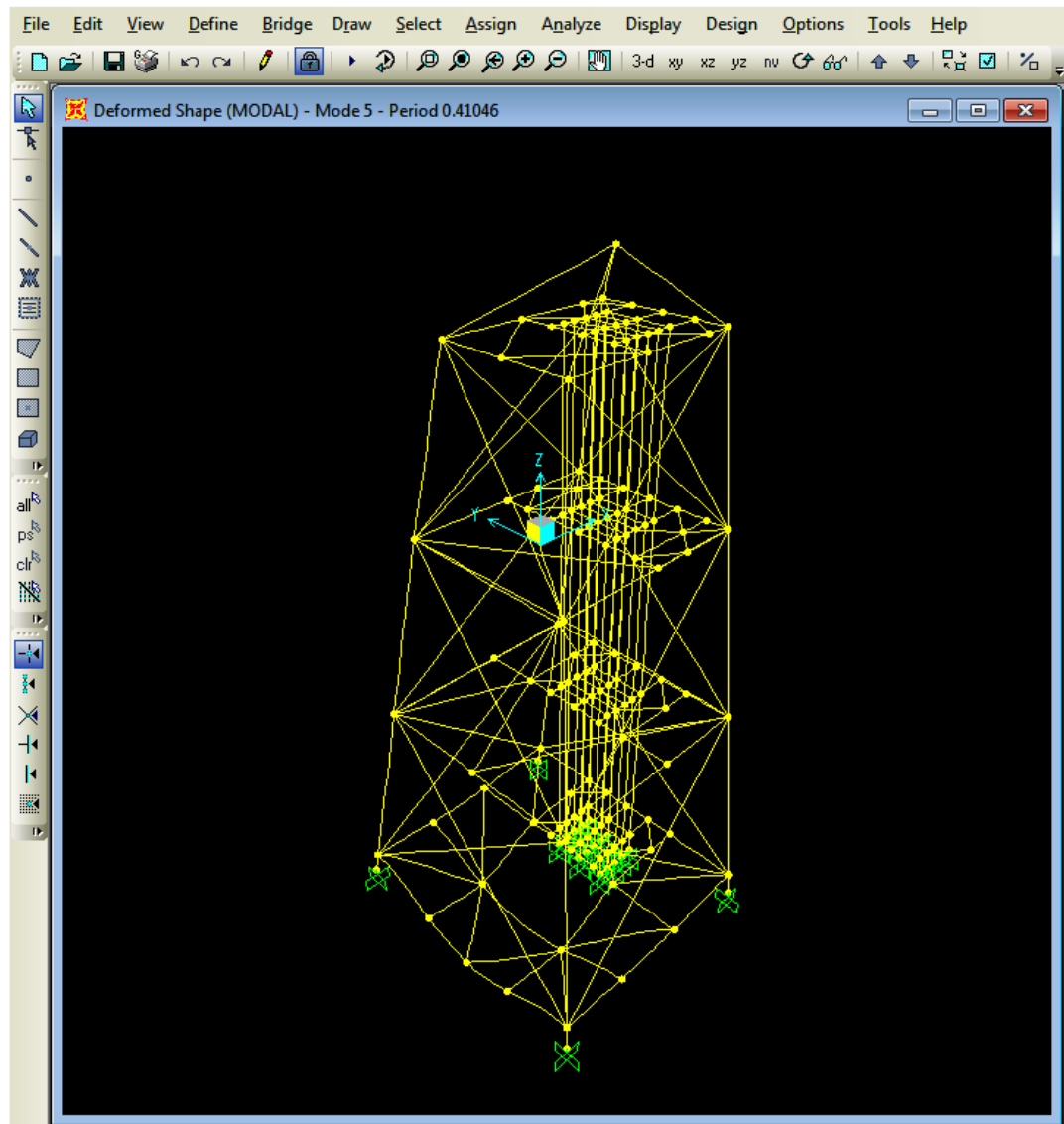
Modal Plots











CURRICULUM VITAE



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