ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

DESIGN OF AN INDUSTRIAL STEEL BUILDING ACCORDING TO NEW TURKISH SEISMIC AND STEEL DESIGN CODES

M.Sc. THESIS

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Department of Civil Engineering

Structure Engineering Programme

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ISTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

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Mohammad zaher Serdar, a M.Sc. student of İTU Graduate School of Science Engineering and Technology student ID 501151075, successfully defended the thesis entitled "**DESIGN OF AN INDUSTRIAL STEEL BUILDING ACCORDING TO NEW TURKISH SEISMIC AND STEEL DESIGN CODES**", which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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To my family for their unlimited support, mom, dad, sister, brothers, aunt & my grandmother for believing in me and offering help throughout this journey

To my future wife & children

To my original country Syria, which I hope I could use the knowledge that I aquired to rebuild it and enlight its people

To my new country Turkey, which gave me a lot and accepted me, May this thesis be a step in repaying its debt on me,

FOREWORD

I would like to thank Doç.Dr. Barlas Özden Çağlayan for taking me under his supervision and giving me the chance to learn from him and having patience on me, I was really lucky and honored to do my thesis with him, such a noble and great person, and I hope I have never disappoint him and that I will make him proud of me.

December 2018 Mohammad Zaher SERDAR
(Civil Engineer)

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ABBREVIATION

FEMA **:** The Federal Emergency Management Agency
AISC **:** The American Institute of Steel Construction
ASCE **:** The American Society of Civil Engineers
HHT **:** Hilber-Hughes-Taylor method
ELM **:** Effective Length Method
DM **:** Direct analysis Method
KISS **:** Keep It Simple and Stupid
IO **:** Immediate Occupancy performance level
LS **:** Life Safety performance level
CP **:** Collapse Prevention performance level

SYMBOLS

$\boldsymbol{Fsr}$	: Allowable stress range, (MPa)
$\boldsymbol{F_{th}}$	: Threshold allowable stress range, maximum stress range for indefinite design life (MPa)
$\boldsymbol{Cf}$	: Constant according to fatigue category
$\boldsymbol{n_{SR}}$	: Number of stress range fluctuations in design life
$\boldsymbol{N_i}$	: The national load to be applied at level i
$\boldsymbol{Y_i}$	: The gravity loads applied at level i
$\boldsymbol{\alpha}$	: Factor depending on the combinations used $\alpha = 1$ for (LDRF), $\alpha = 1.6$ for (ASD)
$\Delta_{\boldsymbol{2nd}}$	: Second-order inter-story drift due to the LRFD or ASD load combinations
$\Delta_{\boldsymbol{1st}}$	: First-order inter-story drift due to the LRFD or ASD load combinations
$\boldsymbol{EA}^*$	: Reduced axial stiffness
$\boldsymbol{EI}^*$	: Reduced flexural stiffness
$\boldsymbol{\tau_b}$	: Factor to account for further reduction in flexural stiffness if the element subjected to high axial load
$\boldsymbol{P_r}$	: Load resisted by element
$\boldsymbol{P_y}$	: Yielding load of the element
$\boldsymbol{f}$	: Scaling factor
$\boldsymbol{SA^{target}}$	: Spectral acceleration of the target spectrum
$\boldsymbol{SA^{record}}$	: Spectral acceleration of the record (unscaled) spectrum
$\boldsymbol{T_n}$	: Natural period of the structure
$\boldsymbol{\bar{A}_i}$	: Target spectral acceleration
$\boldsymbol{A_i}$	: Record's (unscaled)spectral acceleration at i^{th} spectral period
$\boldsymbol{S}$	: Snow load
$\boldsymbol{U_i}$	: Snow load shape coefficient
$\boldsymbol{C_e}$	: Exposure coefficient
$\boldsymbol{C_t}$	: Thermal coefficient
$\boldsymbol{S_k}$	: Characteristic value of snow on the ground at the relevant site

$V_{b,0}$: "The characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights."

$V_m(z)$: Velocity variation with height

$C_r(z)$: Terrain roughness

$C_0(z)$: Orography factor

K_I : Turbulence factor

σ_v : Standard deviation of the turbulence

$I_v(z)$: Turbulence intensity

$q_p(z)$: Peak velocity pressure

ρ : Air density = 1.25 kg/m^3

w : Wind pressure at surface

C_s, C_d : Structural factors and equal to (1) in structures with height less than (15) m

b : Crosswind dimension

h : Building height

R : Response modification coefficient

Ω_0 : Overstrength factor

C_d : Deflection amplification factor

I_e : Importance factor

Ry : Ratio of the expected yield stress to the specified minimum yield stress, F_y

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DESIGN OF AN INDUSTRIAL STEEL BUILDING ACCORDING TO NEW TURKISH SEISMIC AND STEEL DESIGN CODES

SUMMARY

This project is intended to provide guidelines about the advanced methods adopted in the new Turkish codes and other foreign codes such as European and American codes related to the steel structures like (En 1993 , AISC3xx's-16) into practical simplified applications, it might be modified later to serve as manual or courses to help spreading the understanding of these methods and ease their practice in designing offices, and not only mentioning the application steps but also providing the main theoretical knowledge as simplified as possible so that experienced engineers can know each step's reasons and what it represents with little or no need to refer to complex scientific papers or huge technical reports which most practicing engineer avoid out of fear of getting lost, losing valuable time or misinterpret.

In this study it was used a single-story industrial building which serve as the basis for testing and implanting the following topics:

1-Fatigue design of the elements where presented through two approaches one is loads amplification and the other one S-N curves, with some explanation about stress threshold, crack propagation and the mechanism of failure, and later on an example of implementing both of them on the crane supporting beam was presented where the S-N curves proved to be more economical while being more accurate since it is supported by clear scientific reasoning and experimental researches.

2- Direct analysis Method despite being considered as the main analyzing method in the American AISC codes but it still less common in practice compared to Effective Length Method (which is still the main method taught in many universities like Istanbul.

Technical University, despite all its defects presented in this thesis), Direct analysis Method was explained focusing on its significance and why it should be adopted , comparing it with other methods like Amplified First Order Method and Effective Length Method (mainly compared ELM), these explanation included notional loads application to the structures to account for (geometric imperfection , out of plumbness) , reduced stiffness for both axial and bending and their modifications to account for (residual stresses) and second order effects (P-Δ and P-δ) to account the for the element and whole structure deflections.

Also, emphasizing the defects in ELM through missing moment since it fails to consider the second order effects and resulting moments that exist and affecting not only at single element but transferred to other neighbored elements and connections between them, K values problems and the inappropriate considerations resulting from them and the effect of leaning columns which cause weakness to the main columns supporting the structure

and can lead to severe situations if wrong distribution of the loads occurs which may lead to collapse of the whole structure in far less than expected capacity, also comparing with old method which based on substituting the boundary conditions of the element with suitable springs (rotational and linear) and proved the inaccuracy of the effective length method.

3-Selecting and scaling of ground motions:

Some background on seismology and tectonic plates theory was provided with basis for selecting earthquakes accelerations records with suitable characteristic and site properties for scaling procedure comparing between several methods and choosing the most suitable one to scale 8 signals in spectral domain to the MCE defined according to the site and structure properties to achieve defined criteria later in performance based design, also it was mentioned about spectrum matching method which was avoided because it manipulates the frequency content and may lead to change of the properties of the earthquake in order to produce the matched records.

Also, some points or strange values for Kocaeli earthquake has been found that may need further consideration and investigating in the future, which may impose different spectrum definition to be introduced in the code or need spectrum that is developed to account for local earthquakes records' nature.

4- Time History analysis: (scenario-based approach) after selecting and sizing the elements for normal and lateral loads using direct analysis method and defining the plastic hinges to the element according to their anticipated behavior and related performance limits, time history analysis was used to assess the performance of the structure and assure it meets the expectations under the previously selected and scaled accelerations records by direct integration methods and a comparison between Newmark Method and HHT (Hilber-Hughes-Taylor) method with different values of alpha has been done, observing the iteration and converging behavior of the solution at several suggested values and providing some recommendations about the best ways to choose the appropriate value to save time, improve the accuracy and reduce the computational demands of the analysis.

5- Performance based design:

The criteria and definition representing the intended performance was IO immediate occupancy level under time history analysis (scenario-based approach) of the selected signals which was scaled to MCE 2% in 50 years (2500 occurrence earthquake) after several modifications was achieved, and suggestions was provided on how to improve the performance of the structure based on tests of many different arrangements of the model structure.

6- Plastic design was applied for elements involved in the lateral forces resisting system with applying A341-16 code rules for sections thickness and KISS method for load distribution (due to the code restrictions for gusset plates) and according to the forces applied to the surrounding elements when the plastic hinges develop at the expected locations to ensure the correct behavior and energy dissipation mechanism is achieved in the structure.

7- Finite element analysis of the base plate with several arrangements was investigated proving that the common practice and arrangement of the anchors is not correct and need further investigation and research to find the suitable arrangement that allow equal distribution of the loads on the anchors.

8- En-1991-3 and En-1991-4 were used to account for wind and snow loads.

9- Some advanced topics that need further research in future studies:

a-Kocaeli earthquake spectrum.

b-Base plate best arrangement.

c-HHT values and their solution behavior.

DESIGN OF AN INDUSTRIAL STEEL BUILDING ACCORDING TO NEW TURKISH SEISMIC AND STEEL DESIGN CODES

ÖZET

Bu proje, yeni Türk kanunlarında kabul edilen gelişmiş yöntemler ve pratikte basitleştirilmiş uygulamalara (En 1993, AISC3xx-16) gibi çelik yapılarla ilgili Avrupa ve Amerikan kodları gibi diğer yabancı kurallar hakkında kılavuz bilgiler sağlamayı amaçlamaktadır. daha sonra, bu yöntemlerin anlaşılmasını kolaylaştırmak ve ofisleri tasarlamada uygulamalarını kolaylaştırmak için manuel veya kurs olarak hizmet etmek ve sadece uygulama adımlarını değil aynı zamanda uzman mühendislerin her adımın nedenlerini bilmesi için mümkün olduğunca basitleştirilmiş ana teorik bilgiyi sağlamak. ve en çok mühendisin kaybolan korkudan, değerli zaman kaybetmekten veya yanlış yorumlamadan kurtulduğu karmaşık bilimsel makalelere veya devasa teknik raporlara atıfta bulunma gereği yoktur.

Bu çalışmada, aşağıdaki konuları test etmek ve implante etmek için temel teşkil eden tek katlı bir sanayi binası kullanılmıştır:

1- Yorulma tasarımı:

İki yaklaşımla sunulan elemanların yorulma tasarımı, yük eşiği, çatlak yayılımı ve başarısızlık mekanizması hakkında bazı açıklamalarla birlikte yük yükseltmesi ve diğeri SN eğrileridir ve daha sonra bunların her ikisini de uygulama örneğidir. açık bir bilimsel akıl yürütme ve deneysel araştırmalar tarafından desteklendiği için, SN eğrilerinin daha ekonomik olduğu ve daha doğru olduğu ispatlandı.

2- Doğrudan analiz Metot:

Doğrudan analiz Metot Amerikan AISC kodlarında ana analiz metodu olarak kabul edilmesine rağmen, halen daha uzun süredir geçerli olan Etkili Süreç Metodu ile kıyaslandığında (ki hala İstanbul gibi birçok üniversitede öğretilen ana yöntemdir).

Teknik Üniversite, bu tezde sunulan tüm kusurlara rağmen), Doğrudan analiz Yöntemi, önemine ve niçin benimsenmesi gerektiğine, Amplifiye İlk Mertebe Yöntemi ve Etkin Uzunluk Metodu (esasen ELM ile karşılaştırılarak) gibi diğer yöntemlerle karşılaştırılarak açıklanmıştır. Açıklamada, (geometrik kusur, erime dışında), eksenel ve eğilme için azaltılmış sertlik ve (kalıntı gerilmeler) ve ikinci mertebeden etkilerin (P-Δ ve P-δ) hesaba katılması için yapıları dikkate alan yükler uygulanmıştır. eleman ve tüm yapı sapmalarını hesaba katmak.

Ayrıca, ikinci mertebe etkilerini ve sadece tek bir unsuru etkilemekle kalmayıp, diğer komşu elementlere ve bunların arasındaki bağlantılara da etki ederek, ortaya çıkan momentleri dikkate almadığı için, eksik anı ile ELM'deki kusurları vurgulayarak, K değerleri problemleri ve uygun olmayan değerlendirmelerle sonuçlanır. onlardan ve

yapıyı destekleyen ana sütunlara zayıflık veren eğik kolonların etkisi ve yükün yanlış dağılımı meydana geldiğinde, tüm yapının beklenen kapasiteden çok daha az sürede çökmesine neden olabilecek ciddi durumlara yol açabilir. elemanın sınır koşullarının uygun yaylarla (dönme ve doğrusal) ikame edilmesine dayanan ve etkili uzunluk yönteminin yanlışlığını kanıtlayan eski yöntem.

3-Yer hareketlerinin seçilmesi ve ölçeklenmesi:

Sismoloji ve tektonik plakalar teorisine ilişkin bazı bilgiler, çeşitli metotları karşılaştırarak ölçeklendirme prosedürü için uygun karakteristik ve saha özelliklerine sahip deprem hızlandırması kayıtlarının seçilmesi ve spektral alandaki ölçek 8'e göre en uygun olanın 8 taneye göre MCE'ye göre seçilmesi için temel sağlanmıştır. performansa dayalı tasarımda daha sonra tanımlanmış kriterlere ulaşmak için saha ve yapı özellikleri, aynı zamanda frekans içeriğini manipüle ettiği ve eşleştirilen kayıtları üretmek için deprem özelliklerinin değişmesine yol açabileceği için kaçınılmış spektrum eşleme yöntemi hakkında da değinilmiştir.

Ayrıca, Kocaeli depreminin, deprem kayıtlarının doğasını hesaba katmak üzere geliştirilen kod veya ihtiyaç spektrumuna farklı spektrum tanımlaması getirebilecek ileride dikkate alınması ve araştırılması gerekebilecek bazı noktaları veya tuhaf değerleri olduğu bulunmuştur.

4- Zaman Tarihçesi analizi: (senaryo tabanlı yaklaşım) :

doğrudan analiz metodu kullanılarak normal ve yanal yükler için elemanların seçilmesi ve boyutlandırılmasından sonra plastik menteşelerin beklenen davranışlarına ve ilgili performans limitlerine göre elemente tanımlanmasından sonra zaman öyküsü analizi kullanılmıştır. Yapının performansını değerlendirmek ve daha önce seçilen ve ölçeklendirilmiş ivme kayıtları altında beklentileri doğrudan doğruya entegrasyon yöntemleriyle karşılayabilmek ve farklı alfa değerleri olan Newmark Metodu ile HHT (Hilber-Hughes-Taylor) metodu arasında bir karşılaştırma yapıldığını, çözümün yinelenen ve yakınsama davranışını birkaç önerilen değerde gözlemlemek ve zaman kazanmak için uygun değeri seçmenin en iyi yolları hakkında bazı öneriler sunmak, doğruluğu geliştirmek ve analizin hesaplama taleplerini azaltmak.

5- Performansa dayalı tasarım:

İstenen performansı temsil eden kriter ve tanım, birkaç değişikliğe ulaşıldıktan sonra 50 yılda MCE'ye (2500 deprem) MCE'ye% 2 ölçeklendirilen seçilmiş sinyallerin zaman öyküsü analizi (senaryo tabanlı yaklaşım) altında IO doluluk seviyesi ve önerilerdir. Model yapısının birçok farklı düzenlemesinin testlerine dayanarak yapının performansının nasıl iyileştirileceği hakkında bilgi verilmiştir.

6- Dirsekler için A341-16 kod kurallarının uygulanması ve yük dağılımı için KISS yöntemi (köşebent plakaları için kod kısıtlamalarından dolayı) ve çevreye uygulanan kuvvetlere göre yanal kuvvetlere dirençli elemanlar için plastik tasarım uygulanmıştır. Yapılarda doğru davranış ve enerji dağılımı mekanizmasının sağlanabilmesi için plastik menteşelerin beklenen yerlerde geliştikleri unsurlardır.

7- Sünek tasarım:

Baz plakanın çeşitli düzenlemelerle sonlu eleman analizi incelenmiş ve çapaların ortak uygulama ve düzenlemelerinin doğru olmadığı ve yüklerin çapalar üzerinde eşit dağılımına izin veren uygun düzenlemeyi bulmak için daha fazla araştırma ve araştırmaya ihtiyaç duyulduğu kanıtlanmıştır.

8- En-1991-3 ve En-1991-4 rüzgar ve kar yüklerini hesaba katmak için kullanıldı.

9- Gelecekteki çalışmalarda daha fazla araştırmaya ihtiyaç duyan bazı ileri düzey konular:

a-Kocaeli deprem spektrumu.

b-Taban plakası en iyi düzenleme.

c-HHT değerleri ve çözüm davranışları.

1.INTRODUCTION

1.1 History of The Steel Structures

Throughout history humans tried to shelter from natural effects firstly in caves and as the civilization developed they started using different building materials depending on the resources available in the surrounding environments like wood in some cases, and adobes with binders in more developed nations , in general the technics and materials used in building was in some sort reflecting the prosperity of nations and the need to build lasting monuments forced the adopting of new technics and martials.

A breakthrough happened in 1824 when the Portland cement was invented by Joseph Aspdin which paved the way for the reinforced concrete to be introduced into building industry and provide cheaper and less effort demanding structures which led to a booming in all kind of structures but the concrete structures have limited capability in resisting earthquake forces due to weak resistance for tension stresses, cyclic loading and have some limitation due large sections in tall buildings (Url-1, 2018).

Till the 19^{th} century the iron played a secondary role in buildings but due to the technical advancements iron products starts to be employed in some special structures like Southwark Bridge-London (1819) Span 73 m. The longest cast iron bridge, Eiffel Tower (1887 ~ 1889) 9000 tons of wrought iron. (Peter Ball, 2016).

In 1856, Henry Bessemer invented a new method coined to his name which allowed the production of steel in large quantities and much cheaper, and followed by Charles William Siemens who invented the Open-Hearth Furnace, these two inventions allowed the steel to replace the wrought iron as the main structural material, this was further supported by the standardization of steel sections by hot-rolled sections, and the development of new connecting methods (riveting, bolting, and welding) which allowed to fusing and secure the continuity of the elements with minimum workforce (Peter Ball, 2016).

These features allowed the steel structure to become fast deployable pre-engineered and cost effective, furthermore the slight difference between tensile and compressive capacity, high cyclic-loading resistance, and high energy dissipation allowed it to become very effective especially in earthquakes resisting systems.

The technological advancement in the recent years led to introduction of further more kinds and applications for the structural steel with high strength and economically effective prices, allowed types of the painting of that prolong the life time of the structures and protecting it against harmful environments, automatization of production line to obtain high accuracy parts (dimensions, cuts, and drilling) using software and the deep understanding of the structural steel elements behavior under different loading conditions that was developed by researchers around the world.

1.2 Literature Review

1.2.1 Fatigue:

One of the important effects on steel structures that is subjected to high dynamic forces (like bridges etc.) to be considered is fatigue which happens when elements or connections subjected to repeated dynamic and high cyclic loadings of frequency and magnitude sufficient to initiate cracking and progressive failure like (cranes, bridges and offshore structures), even though the stresses much less than yielding failure stress.

Also, it should be noted that fatigue failure is not related only with structures but more with mechanical engineering since it is very crucial in almost all the vehicles so that it imposes maintenance cycles, parts to be checked or replaced and overall worth or effective life-time of the machine , in fact the earliest observation of fatigue phenomena was done by Wöhler on railroad rolling stock where he observed that repeated loading resulted in failure even though the stresses in the material was well below the yielding strength(Wöhler, 1870).

Fatigue phenomena happens in metals in general but different types of metals have different behavior for example structural steel has endurance limit (which mean a stress threshold no matter how many cycles are as long as the stress is below this threshold

fatigue failure will not occur Fth) while Aluminum don’t have such limit as illustrated in the Figure 1.1 (Dan Dubina, 2012, F.C.Campbell 2008).

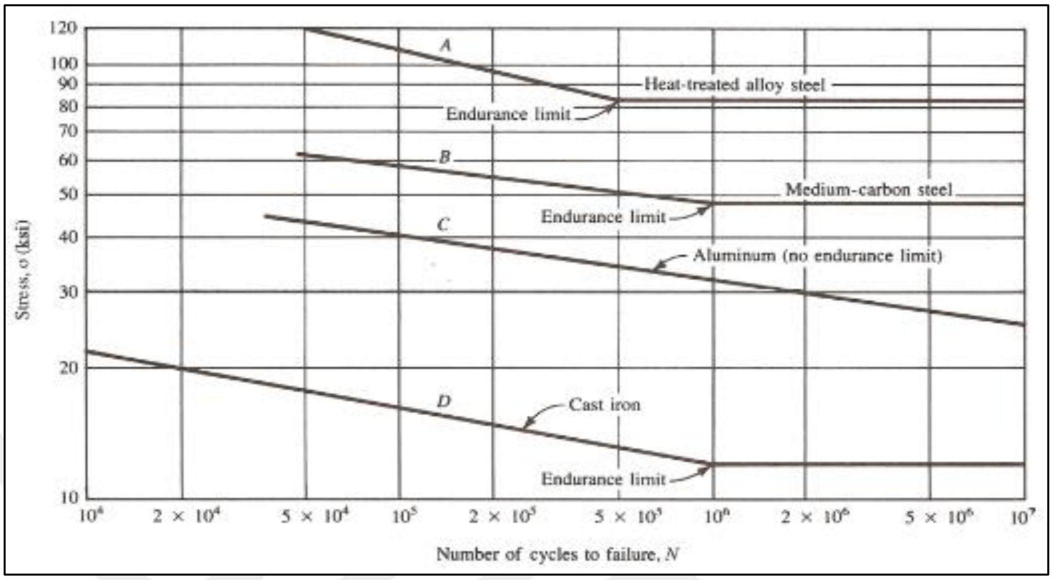


Figure 1.1 Several material S-N relations (F.C.Campbell 2008).

This feature is related to the steps of fatigue failure:

1-Crack initiation.

2-Cracks propagations.

3-Fracture.

The macro mechanism of fatigue is briefly explained as follow:

1-Crack initiation : the crack usually starts at localized stress concentration points like material discontinuity , notches ,bolt holes… this concentrated stress raises above yielding stress (even the average stress is still under yielding strength) so the plastic strain results in each cyclic loading as the a result a slip occurs till the plastic deformation is no longer possible so a notch occurs or in other word the material loose its continuity (to reach this point the stress should go above stress range threshold).

2-Crack propagation: the repeat of the load cycles causes the propagations of the crack from one grain to another and as the crack size increases the resistance of the applied stresses decreases till it reaches appoint the element no longer can sustain the applied forces.

3-Fracture: as a result of crack propagation the element fail in sudden fracture (Gopinath and Mayuram, 2014).

According to (Fisher et al., 1970; Fisher et al.1974) Structural steel with minimum yield strength of 250 to 690 MPa doesn't behave differently toward fatigue strength.

It also should be noted that the crack propagation is affected by temperature and the surrounding environment such as exposed to harsh chemicals.

Despite the advancement in fracture mechanic which may allow a more accurate solution the common practice that is adopted in codes is S-N curves where N represent the number of cycles to failure and S is the stress range, these curves where obtained by extensive database developed in United States and abroad (Keating and Fisher 1986) and (Cf) is a coefficient were used which correspond to 2.5% probability of failure during design life:

$$Fsr = 6900 * \left(\frac{Cf}{n_{SR}}\right)^{0.333} \geq F_{TH} \quad \textbf{(1.1)}$$

$$n_{SR} = (number\ of\ stress\ fluctuations\ per\ day) * 365 * (years\ i\ design\ life) \quad \textbf{(1.2)}$$

And these relations for different values of Cf according to the category are represented in the Appendix 3 of A360-16 and its commentary as in figures (1.2&1.3).

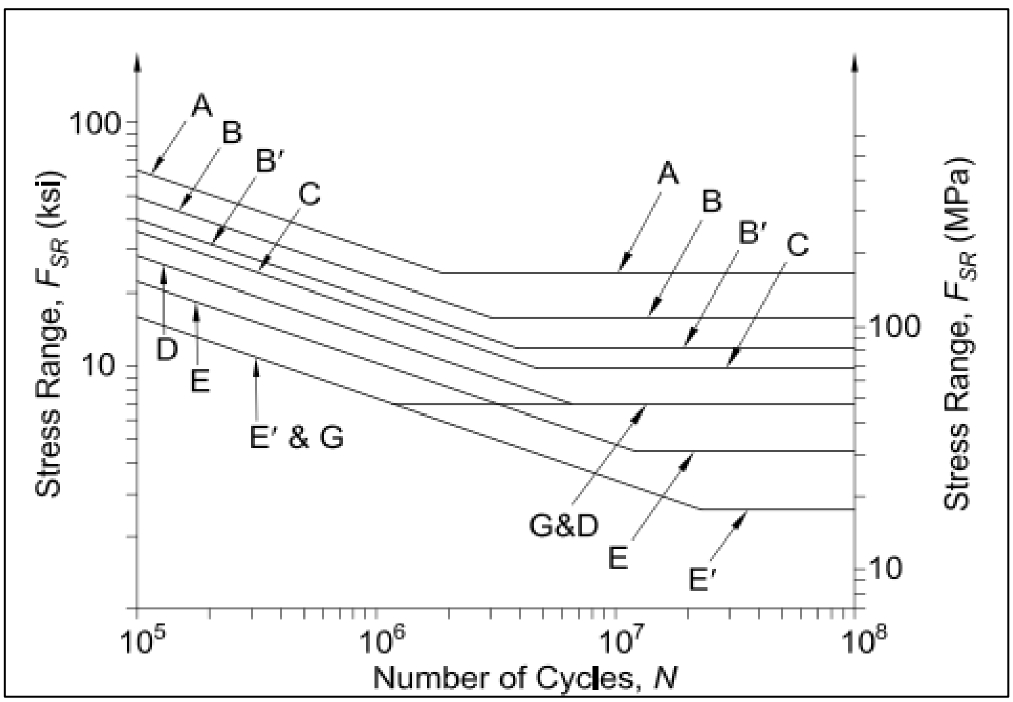


Figure 1.2:S-N Curves part 1. (A360-16 Appendix 3).

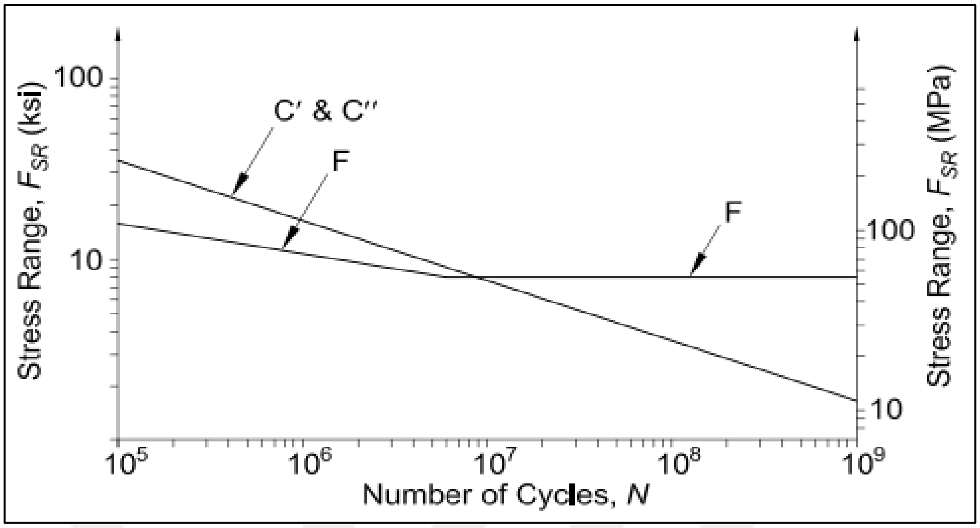


Figure 1.3: S-N Curves part 2. (A360-16 Appendix 3).

To use these curve engineer should refer to A360-16 Appendix 3 to choose the suitable category for his specific elements and joints under consideration.

Although it should be noted that a building that fatigue shouldn't considered for wind loads effects or seismic effects on typical lateral force-resisting systems in accordance with A360-16 Appendix 3.

And since compressive stresses don't initiates cracks so if the effects only produce compressive stresses there is no need to consider fatigue, but in case the cyclic load causing changing in stresses between compressive and tensile stresses the difference between the maximum compressive and maximum tensile would be taken as stress range to be compared with the allowed stress range.

The other approach: that is mentioned in the literature is to amplify the loads used in design as mentioned in (Din 120,1949), this method is depending on the engineer judgement to the nature of the use and its intensity, using this estimation engineer will classify the crane into one of four groups and according to this classification will multiply the applied loads with suitable factor.

1.2.2 Designing for stability

Relaying on the current specifications it is noted that the main methods for analyzing and accounting for stability are:

1-Amplified 1^{st} order method.

2-ELM effective length method.

3-DM direct analysis method.

We will focus on (ELM) and (DM) since they are the most used methods in practice and more accurate but as a summary, we will mention the main differences between the three methods as shown in the Table1.1 and later on we will demonstrate some differences in the results.

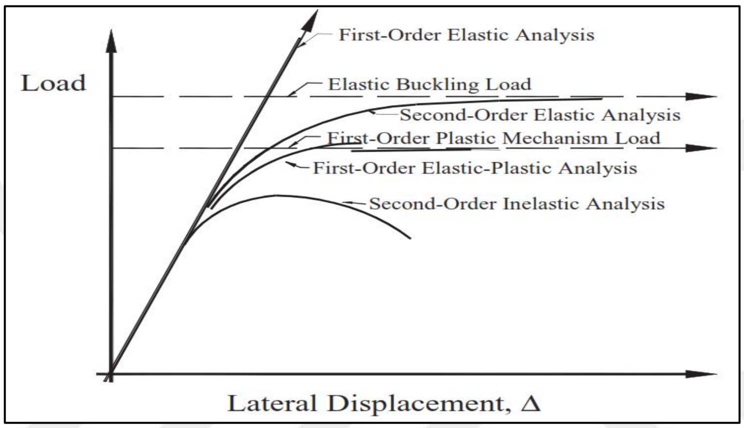


Figure 1.4: Comparison between load-displacement curves of several analysis approaches (L. Geschwinder 2002).

In the recent specifications (starting from 2010 A360-10) the Direct Analysis Method has been moved to chapter C so it become the main method for designing for stability while the two other methods have been moved to the Appendix 7 as alternative methods, even though the most common method in the industry is the Effective Length Method.

The main features for the Direct analysis method are:

1-It accounts for the destabilizing effects of initial geometric imperfections such as out-of-plumbness by applying notional loads or include it in model (notional loads are far simpler approach).

2-It accounts for residual stresses from rolling during manufacturing by reducing flexural and axial stiffnesses of the elements.

3-P-δ and P-Δ effects through 2nd order analysis.

4-Using K=1 in all situations eliminating a lot of mistakes in practice.

5-It is more convenient to be adopted in computer program than other methods.

6-It is applicable for all types of structural systems like (moment frames, space frame and combined systems).

7-Capture the internal forces more accurately and provide the most accurate simulation to the real element behavior.

8-Lead to correct design of beams and connections providing rational constrains to columns.

9-Applicable to all side way amplifications values (no limit on $\Delta_{2nd}/\Delta_{1st}$).

Table 1.1: Differences between the main analyzing methods (AISC design guide 28).

	Direct Analysis Method (DM)	Effective Length Method (ELM) (See Note 5)	First-Order Analysis Method (FOM)
AISC *Specification* Reference	Appendix 7	Section C2.2a	Section C2.2b
Limitations on the Use of the Method	None	$\Delta_{2nd}/\Delta_{1st}$ or $B_2 \le 1.5$	$\Delta_{2nd}/\Delta_{1st}$ or $B_2 \le 1.5$, $\alpha P_r/P_y \le 0.5$
Analysis Type	Second-order elastic (See Note 1)	Second-order elastic (See Note 1)	First-order elastic
Structure Geometry in the Analysis	Nominal (See Note 2)	Nominal	Nominal
Notional Loads in the Analysis (See Note 3)	$0.002Y_i$ Minimum if $\Delta_{2nd}/\Delta_{1st} \le 1.5$ Additive if $\Delta_{2nd}/\Delta_{1st} > 1.5$ (See Note 2)	$0.002Y_i$ Minimum (See Note 2)	$2.1(\Delta/L)Y_i \ge 0.0042Y_i/\alpha$ Additive (See Note 6)
Member Stiffnesses in the Analysis	Use $EA^* = 0.8EA$ Use $EI^* = 0.8\tau_b EI$ $\tau_b = 1.0$ for $\alpha P_r/P_y \le 0.5$ $\tau_b = 4[(\alpha P_r/P_y)(1 - \alpha P_r/P_y)]$ for $\alpha P_r/P_y > 0.5$ (See Note 4)	Use nominal *EA* and *EI*	Use nominal *EA* and *EI*
Design of Individual Members	Use Chapters E, F, G, H and I, as applicable	Use Chapters E, F, G, H and I, as applicable	Use Chapters E, F, G, H and I, as applicable
	Use $K = 1$ for calculating member strengths	Determine K for calculating member strengths from sidesway buckling analysis (Can use $K = 1$ for braced frames; can use $K = 1$ when $\Delta_{2nd}/\Delta_{1st} \le 1.1$)	Use $K = 1$ for calculating member strengths
	No further member stability considerations	No further member stability considerations	Apply amplification $B_1 = C_m/(1 - \alpha P_r/P_{e1}) \ge 1$ to beam-column moments

1.2.2.1 Notional loads:

Notional loads are lateral loads that are presented as a ratio of the gravity loads that are applied to the same level in the structure as shown in Figure 1.5, and since it represents the geometric imperfection so its ratio is derived from the assumed out of plumbness specified in the applied code for example in AISC codes out of plumbness ratio equal (1/500) so the national loads coefficient is 0.002 or:

$$N_i = 0.002 * \alpha * Y_i \quad \textbf{(1.3)}$$

N_i : Notional load to be applied at level i.

Y_i : Gravity loads applied at level i.

α : Factor depending on the combinations used $\alpha = 1$ for (LDRF), $\alpha = 1.6$ for (ASD).

Also, an additional $N_i = 0.001 * \alpha * Y_i$ can be add to account for influence of residual stresses as a simplification (this method is not necessary in computer program since τ_b is automatically calculated and explained more later in the next section).

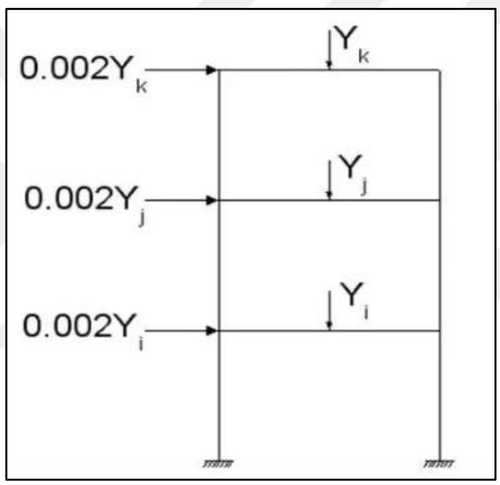


Figure 1.5: Explain the application of national loads (AISC Education).

Also, we can model the out of plumbness instead of applying national loads but this is impractical approach.

In case of special structures where levels hard to be considered the notional loads is applied at each location where gravity loads are applied.

Another aspect concerning the national loads is its participation in load combinations, according to AISC specifications notional loads may add to the gravity loads combinations only if the second order average story drifts to the first order story drifts less than 1.7 in case of reduced stiffness and 1.5 in case of unreduced stiffness or:

$$\frac{\Delta_{2nd}}{\Delta_{1st}} \leq 1.7 \quad (reduced \text{ stiffness }) \quad \textbf{(1.4)}$$

$$\frac{\Delta_{2nd}}{\Delta_{1st}} \leq 1.5 \quad (unreduced \text{ stiffness }) \qquad \textbf{(1.5)}$$

Otherwise the national loads should be added to all load combinations (including lateral load combination) in a way and direction that amplify its effects, for simplicity and to avoid applying this check we can apply it directly to all load combinations.

While the new codes started to mandate the use of notional loads even in the Effective Length Method but this step is not enough firstly because as we will see later first order is not enough to capture the real effect, these loads need the application of nonlinear load cases that include all the forces effecting at the same time to give the true behavior and this step is commonly ignored or forgot in practice.

1.2.2.2 Residual stresses

Due to rolling procedure, deferential cooling rates during manufacturing and out of straightness that exists in all rolled shape (within the limits permitted in ASTM A6 specification), a residual stress pattern is assumed as shown in the Figure1.6.

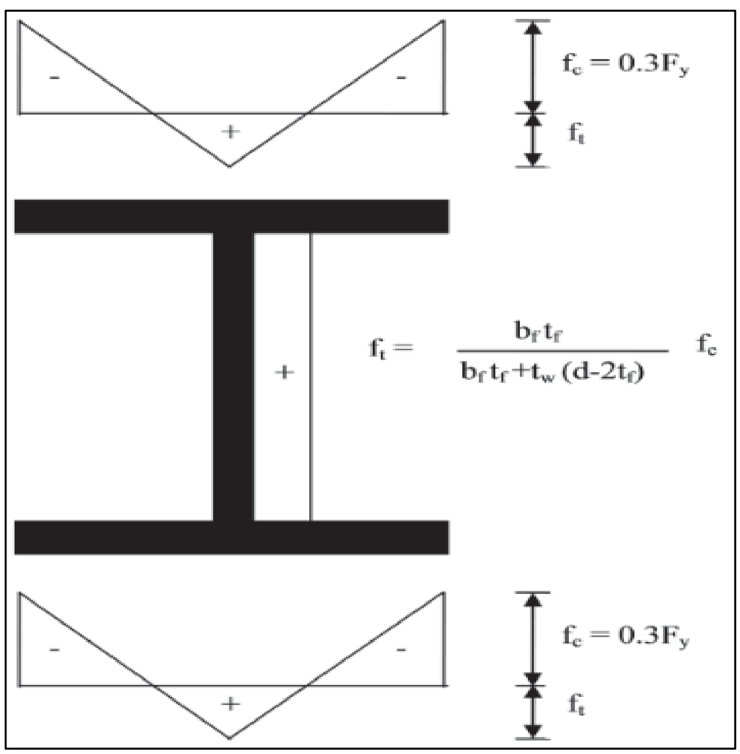


Figure 1.6: Typical assumed residual stresses for rolled wide-flange shapes (Galambos and ketter,1959, (AISC design guide 28)).

The residual stresses cause early yielding to the section which reduce its effectiveness to account for these effects the Direct Analysis Method force a reduction in the axial and flexural stiffness by 0.8 and $0.8 * \tau_b$ respectively as follow:

$$EA^* = 0.8 * EA \tag{1.6}$$

$$EI^* = 0.8 * \tau_b * EI \tag{1.7}$$

Where τ_b is calculated as follow:

$$For \alpha P_r/P_y \leq 0.5 \quad \tau_b = 1 \tag{1.8}$$

$$For\ \alpha P_r/P_y > 0.5 \quad \tau_b = 4 * \frac{\alpha * P_r}{P_y} * (1 - \frac{\alpha * P_r}{P_y}) \tag{1.9}$$

α : as mentioned earlier in notional loads $\alpha = 1$ (LRDF) & $\alpha = 1.6$ (ASD).

As mentioned before instead of applying this check we can directly take $\tau_b = 1$ but we should add a national load equal to $0.001 * \alpha * Y_i$ at each level.

Another aspect to be considered about reduced stiffness is that for serviceability checks we should consider the nominal section not the reduced one.

1.2.2.3 2nd order analysis:

The 2nd order analysis is achieving the equilibrium while in the same time considering the effects of the system displacements and individual members curvatures and deformations, in other word considering P-Δ and P-δ effects and this more explained in Figure 1.7.

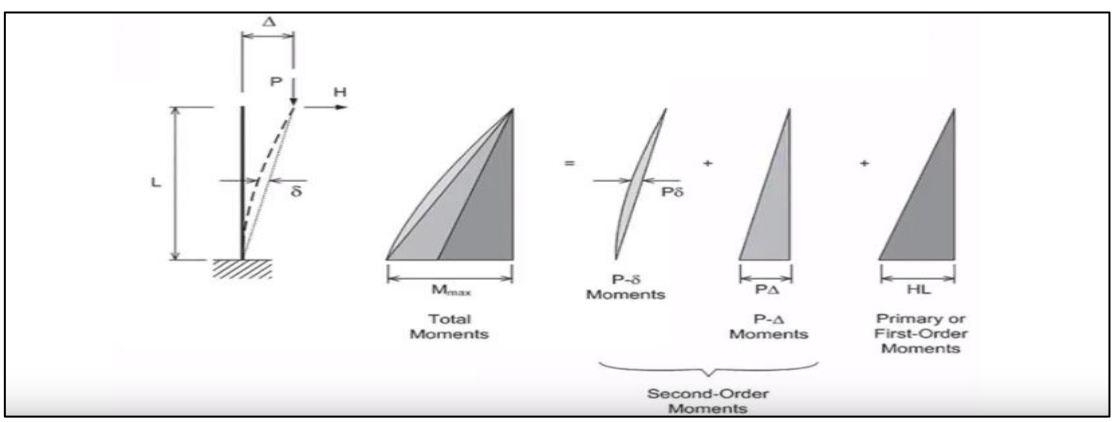


Figure 1.7: First- and second-order effects (AISC design guide 28).

As we can notice there is a considerable difference between the first order moment and second order's one caused by P-Δ and P-δ, so we will discuss them hereafter.

P-δ effect:

The effect produced by loads due to the consideration deflections in the element between its joints, as a result it causes an amplification of the moments (and deflection) mainly by the effect of the axial loads with leverage of the deflection as shown in Figure 1.8.

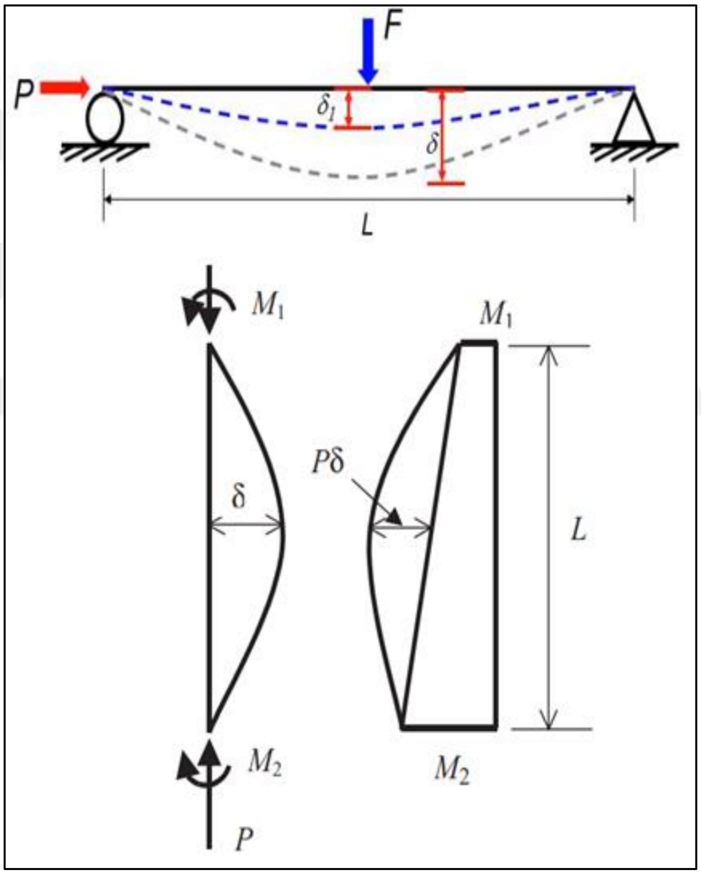


Figure 1.8: P-δ Effect (L. Geschwinder 2002, AISC Education).

To capture these effects in computational model designer should apply meshing to all elements not less than 4 segments for each.

P-Δ effect:

Defined as the effect produced by loads due to the deflections of the whole system like the situation shown in Figure 1.9.

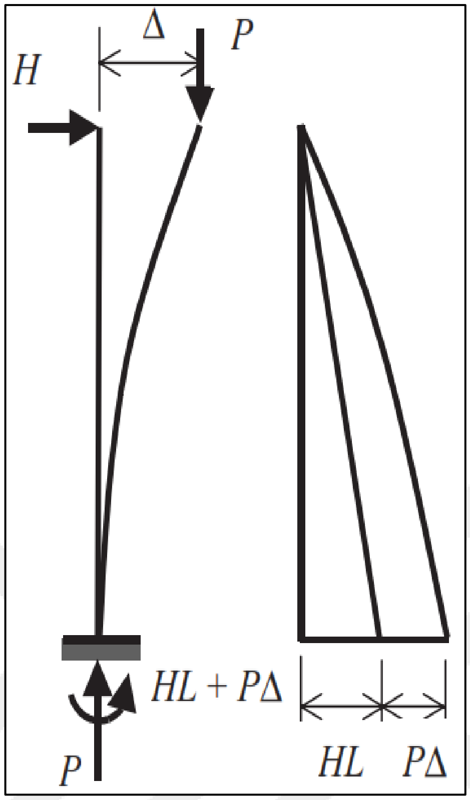


Figure 1.9: P-Δ Effect (L. Geschwinder 2002, AISC Education).

To consider the P-Δ effect load combinations into nonlinear load cases, also it should be noted that when considering P-Δ effect adding separated loads effects isn't enough (as on linear 1st order) to capture the real behavior instead we should consider all participating forces effects in the same time which produce the real accumulated deflections and effects resulting from their application.

1.2.2.4 Defects in Effective Length Method:

Missing moment:

As shown in the Figure 1.10, the Direct method gives a rational column behavior since all columns have some out of plumbness while the Effective Length method represents an ideal behavior where the column fails under axial load without having any moment. So, we can see clearly that both methods result a failure at almost the same axial load but the moment that presented in the DM system will be accounted for in the connections and baseplates while in ELM will result in ideal design that is contains the risk of having moment not accounted for in real life.

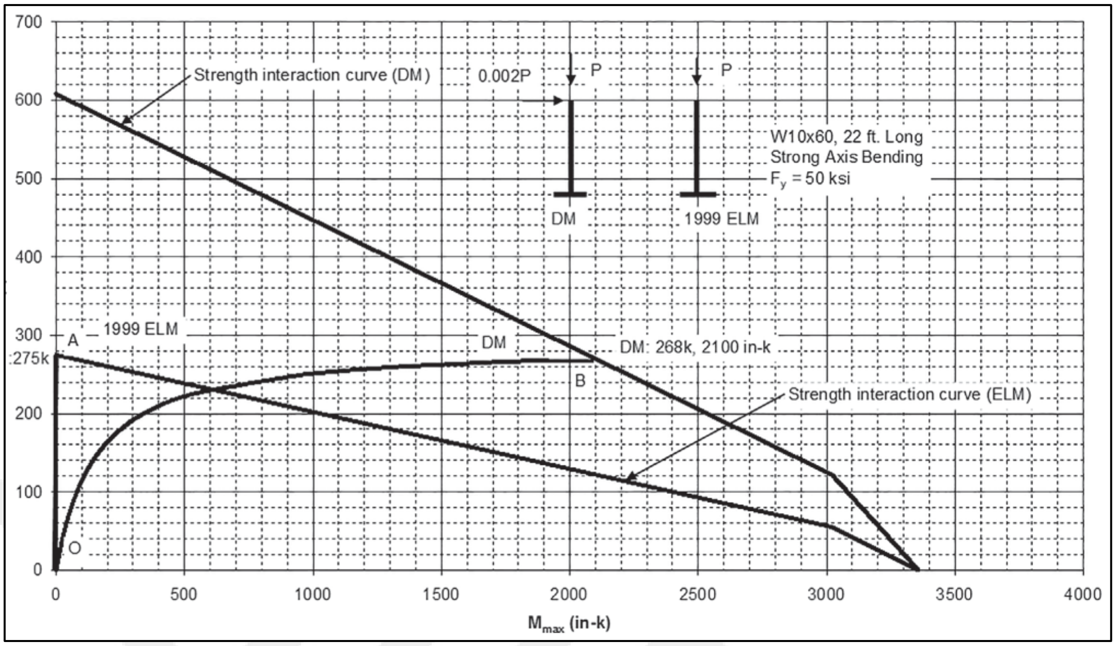


Figure 1.10: DM versus 1999 AISC specification ELM, cantilever column with axial load application to failure (AISC design guide 28).

Even if we applied the recommendations in the later specification and introduced the notional loads to the ELM there will still be some lost moment that would be captured in DM by the second order analysis (P-Δ and P-δ) as shown in Figure 1.11.

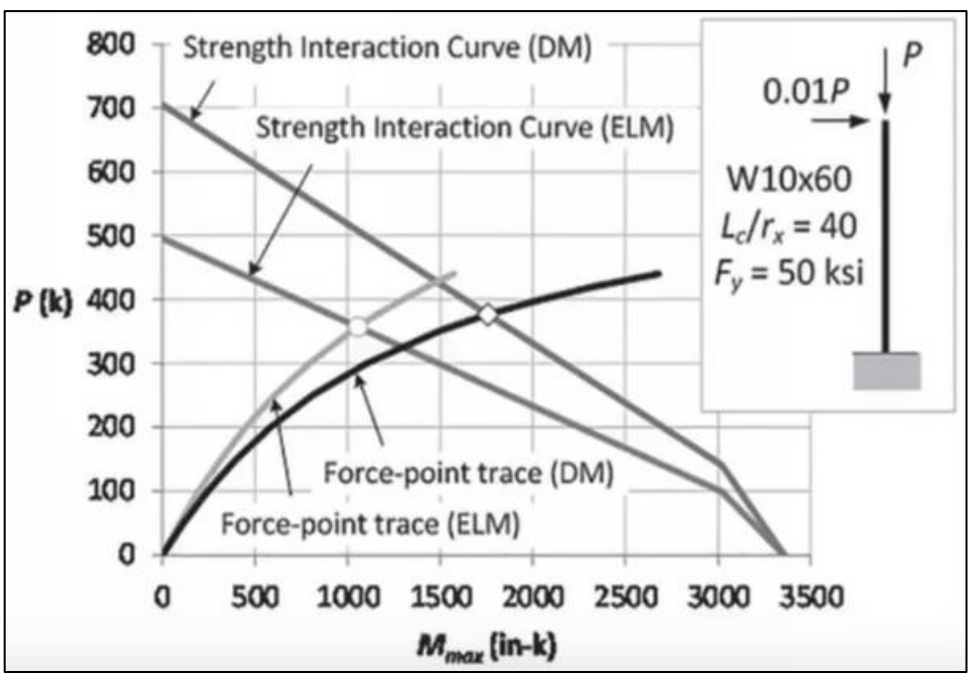


Figure 1.11: DM vs ELM with same notional load considered (AISC design guide 28).

K values:

One of the most confusing aspect about applying ELM is choosing the correct for each element considering its specific situation and the behavior of the connected elements, and to explain the problem more clearly, we will present some of the tests on frame that we created as in Figure 1.12.

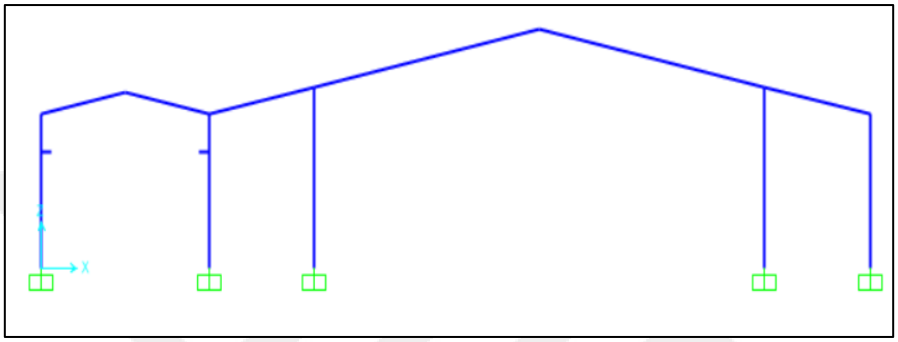


Figure 1.12: Moment frame used to test K.

Eigen Buckling Analysis was applied to this moment resisting frame (noting that all supports are fixed and the connections are rigid) and 2d analysis in the x z plan, while applying a force at the top of each column equal to its load share from the total loads applied to this frame as shown in Table 1.2 and Figure 1.13 & Figure 1.14.

Table 1.2: The participation of the columns.

Column #	Column load	Participation ratio
1	34.93	0.18
2	43.85	0.23
3	42.59	0.22
4	40.28	0.21
5	32.08	0.17
Sum	193.73	1

After applying the EBA we obtained the buckling modes of the structure and the factor related to each mode, and we will focus on the first and third modes to emphasis the problems in K values, so we will show their deformed shape as shown in Figure 1.15 & Figure 1.16 respectively.

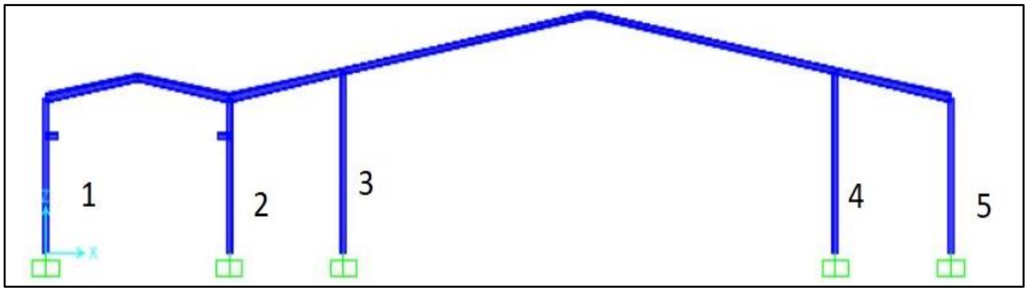


Figure 1.13: Column numbering**.**

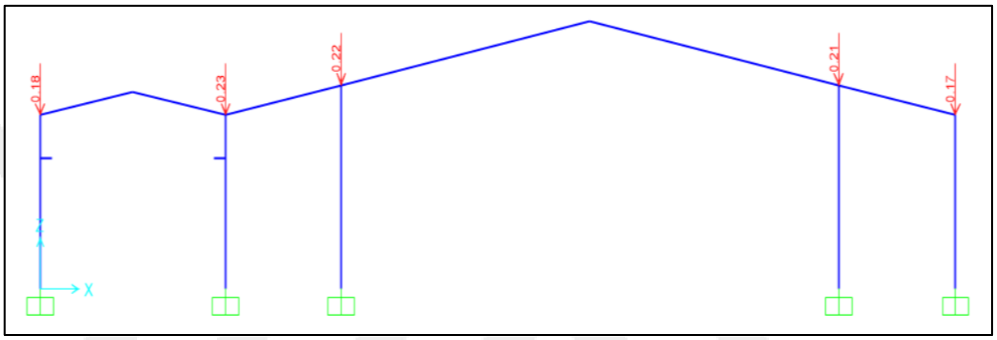


Figure 1.14: Assigned loads shares.

It should be noted that the modes are ordered as the least amount of force needed to cause buckling in structure comes first, buckling factors are shown in Table 1.3.

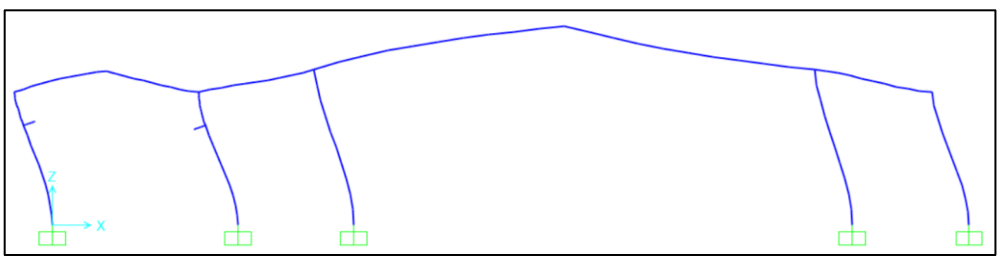


Figure 1.15:1st Buckling mode with buckling factor = 62635.6629.

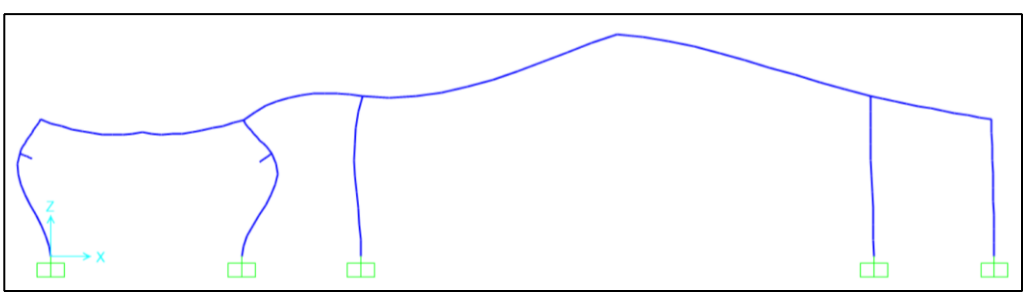


Figure 1.16: 3rd Buckling mode with buckling factor = 246902.6748.

Table 1.3: Buckling modes and their factors.

Output Case	**Step Type**	**Step Num**	**Scale Factor**
Text	Text	Unitless	Unitless
BUCK1	Mode	1	62635.6629
BUCK1	Mode	2	123596.4535
BUCK1	Mode	3	246902.6748
BUCK1	Mode	4	290818.3671
BUCK1	Mode	5	425153.15
BUCK1	Mode	6	465837.248

As we can notice from Figure 1.15 the first buckling mode is representing a situation where the whole structure buckle and the columns K factors associate with this mode are calculated used Euler formulation: (results shown in the Table 1.4).

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} \quad \textbf{(1.10)}$$

It should be mentioned that the 1st buckling mode is the mode governing the actual (or expected) behavior of the building.

Table1.4: K Factors of the columns associate with 1st mode.

Column #	K
1	1.194868
2	1.066435
3	0.923429
4	0.949539
5	1.246815

Also, from Figure 1.16 we can notice that the main part that is buckling is the columns 1,2 and in the same way we can acquire the values of K factors associate with third mode as in Table 1.5.

Table 1.5: K Factors of the columns associate with 3rd mode.

Column #	K
1	0.601821
2	0.537133
3	0.465105
4	0.478256
5	0.627986

As we can see the K factors for the 3rd mode is far less than K values for the 1st mode in other world the building will collapse according to the 1st mode.

Another method was applied to obtain K factors and focusing only on column #1 just to explain the idea, by applying unit load and moment at the top of this column and taking the displacement and rotation associate with each one respectively (the displacement from the load and rotation due to moment) by taking the inverse of them we will have the stiffness resulting from its connection and the effect of other members represented as horizontal and rotational springs as shown in the Figure1.17.

The horizontal spring stiffness is equal to $5681.82\ \frac{1}{kN}$ and the rotational spring stiffness $124192.7\ \frac{1}{kN*m}$.

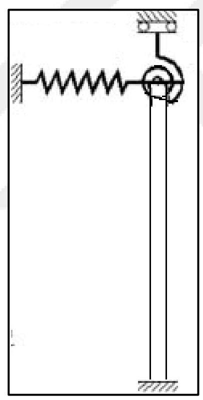


Figure1.17: The equivalent column.

Using the (Çeyse, 1983) shown below in Figure 1.18, Table 1.6 and equations (1.11 & 1.12) we get $k = 0.59 \cong 0.6$ which almost the same as the 3rd mode this mean using these graphs and table (which supposed to take in account the all the effects related to the column) will lead to ill estimation of the real buckling length and the collapsing mechanism.

$$\delta = \frac{c * l^3}{EJ} \quad \textbf{(1.11)}$$

$$\gamma = \frac{k' * l}{EJ} \quad \textbf{(1.12)}$$

Table 1.6: Stiffness calculations to obtain the effective length using Figure 1.18 and equations (1.11 & 1.12).

	force1		moment1
Displacement	0.000176		7.585E-06
Rotation	0.000007585		8.052E-06
Invers (lin. stiff.) (c)	5681.818182	Invers (not used)	131839.16
Invers (not used)	131839.1562	Invers (rot. stiff.) (k')	124192.75
L	9		9
J	0.0006618		0.0006618
E	199950000		199950000
δ=	31.30161095	ϒ=	8.4467681
1/δ=	0.031947238	From Figure1.18	
		=>β(K)=	0.59

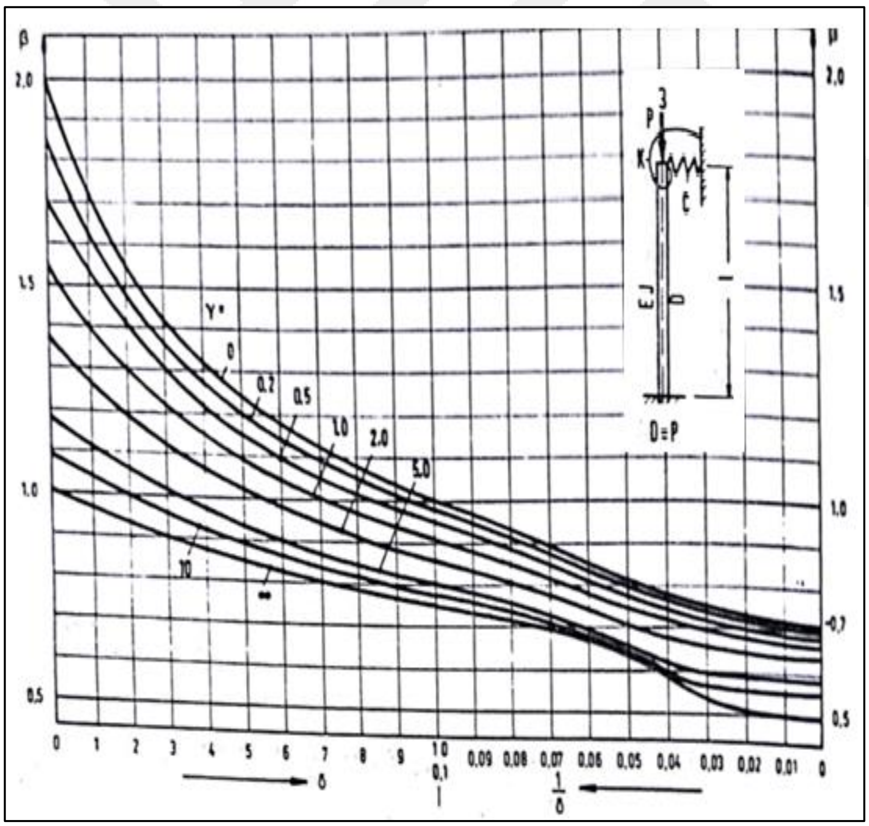


Figure 1.18: Stiffness factors and diagram to obtain effective length (Çeysa,1983).

Leaning columns effect:

The leaning column is a pin ended column that is supported by unbraced moment frame, as in Figure 1.21, where the pin-ended columns actually doesn't participate in the lateral loads resistance but on the opposite amplify the buckling of the moment frame since they rely on them for stability.

To understand its effect, let's consider the structure presented in Figure 1.19, it represents a symmetric frame and it is subjected to symmetrically placed vertical load, both first order and second order analysis will give same the reactions presented in Figure 1.19.

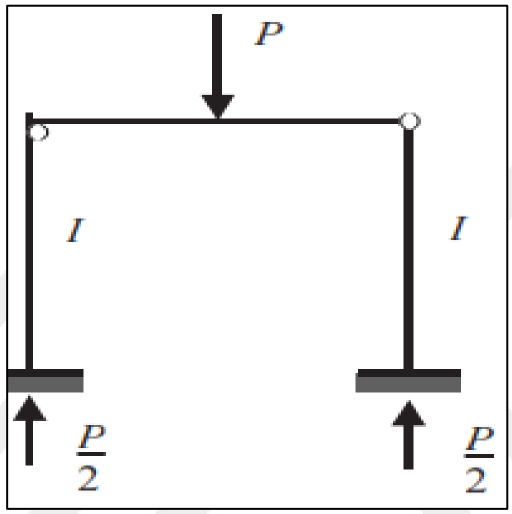


Figure 1.19: Symmetrical structure with vertical load reaction (L. Geschwinder 2002).

Now if we add a lateral load to the structure as shown in Figure 1.20, first order analysis will yield moment as in addition to the axial forces in the support but the moments will further increase in the second order analysis.

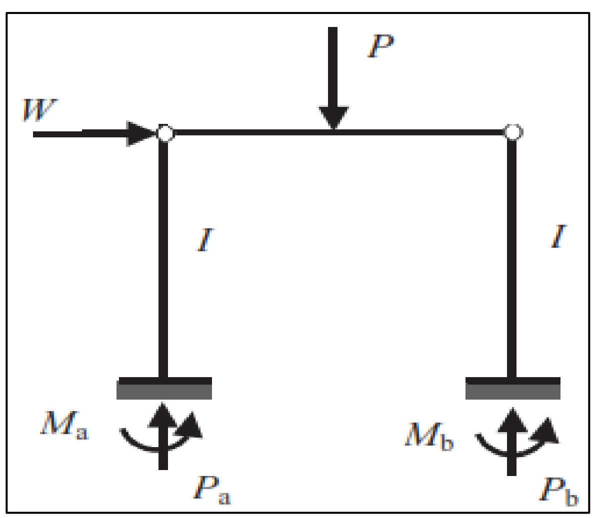


Figure 1.20: Model structure with horizontal load reaction (L. Geschwinder 2002).

Now let's add leaning columns, first we will add a leaning column to the first system (Figure 1.19) as demonstrated in the Figure 1.21, a first order analysis will give the same reactions as the previous system for the moment frame so it appears as leaning columns didn't express any effect, also if the structure subjected to second order analysis the result will remain the same.

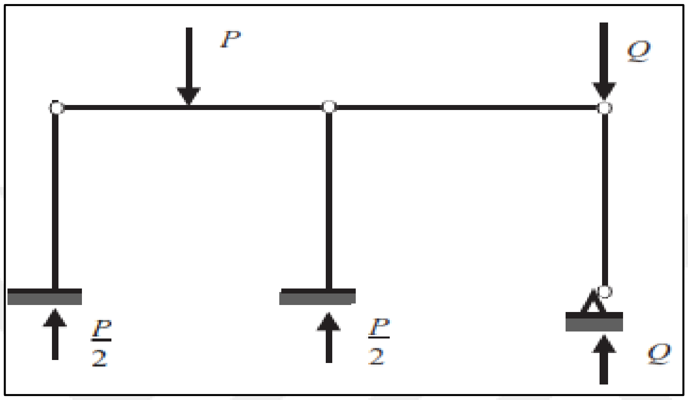


Figure 1.21: Model structure with leaning column vertically loaded only (L. Geschwinder 2002).

Now let's add the leaning columns to the second system (Figure 1.20) with the lateral load existing as in Figure 1.22, a first order analysis will also give the same forces presented in Figure 1.20, as if the leaning columns didn't introduce any differences ,but a second order analysis results will be different than the previous structure because of bending deflections and load displacement interaction the new results will include amplification in the moment due to side sway of the structure and vertical loads (L.Geschwinder 2002).

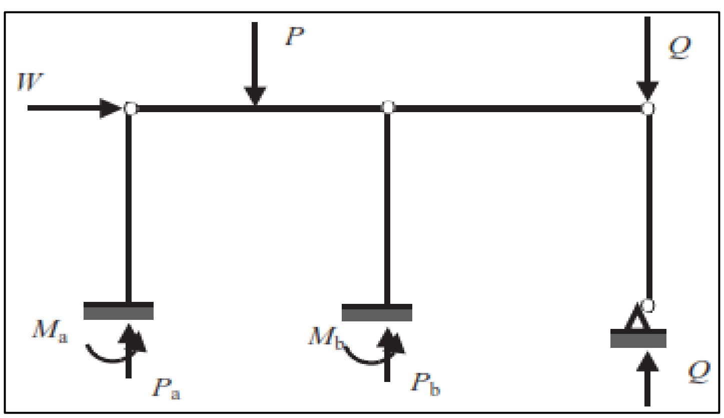


Figure 1.22: Model structure with leaning column and horizontal load reaction (L.Geschwinder 2002).

To further demonstrate these effects, we modeled a basic structure to represent this effect through buckling factors which is obtained by subjecting computer model to eigen buckling analysis.

1st Case: K=2.02 all the load is applied to the main column (the cantilever) its behavior is almost equal to cantilever factor in the codes (actually the same result is obtained when the analysis applied on a cantilever only but we modeled it in this way to emphasis that unloaded leaning column doesn't produce any difference) Figure 1.23.

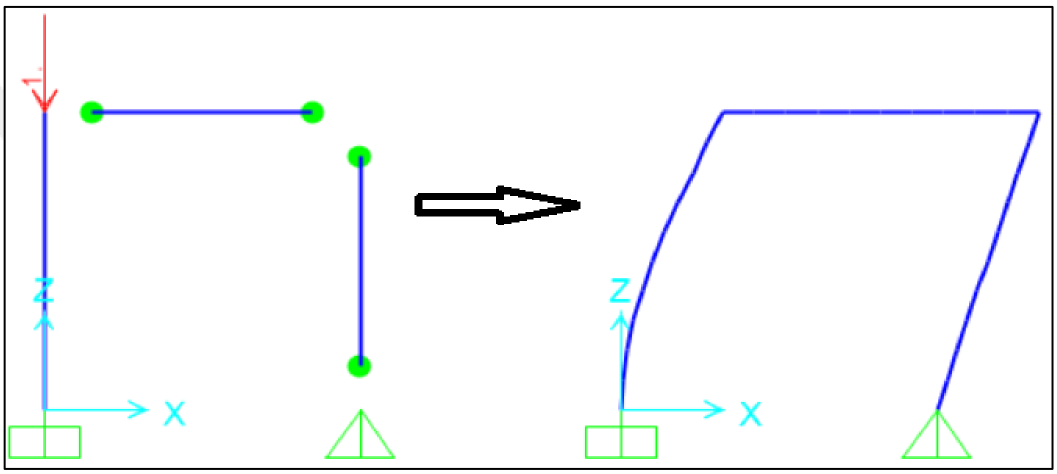


Figure 1.23: 1st Case.

2nd Case: as in Figure 1.24**,** K=2.75 the forces are equally applied on both columns we can observe an amplification of the K factor equal to 1.375.

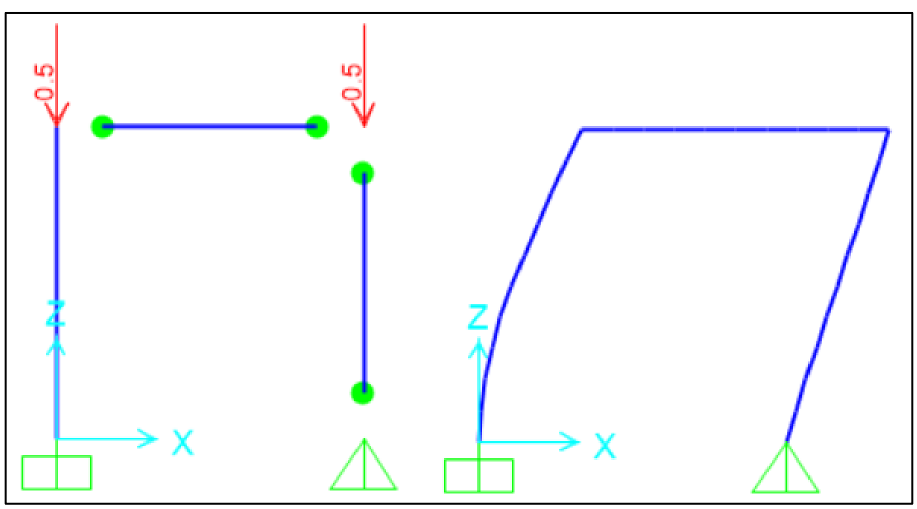


Figure 1.24: 2nd Case.

3rd Case: as in Figure 1.25**,** K=3.81 the forces are applied by ratio 1:3 on cantilever and the leaning column respectively we can observe an amplification of the K factor equal to 1.907.

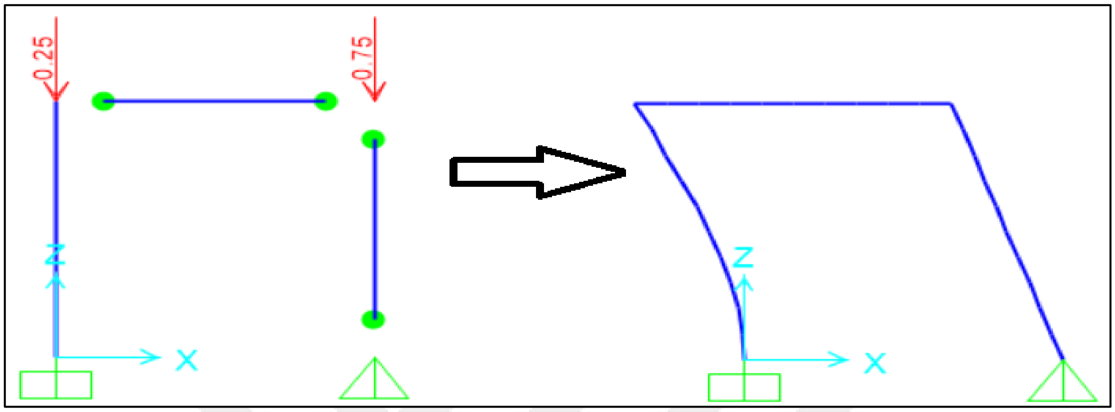


Figure 1.25: 3rd Case.

So, as could be seen that first order analysis miss a lot of important effects on the structures and even the second order need the application of some lateral load (as the notional loads) to correctly interrupt and consider the effects on the structure like the leaning columns which imposes a real threat for the stability of the structure and can be highly missed or not defined properly by commonly imposed K factor mentioned in the codes, since as we saw before the buckling in not an element oriented problem only but rather the whole system problem.

Note that as a matter of fact it is enough to model one leaning column with applied force equal to the sum of the forces on all other leaning columns.

So, during design of frames designer should consider the influence of leaning columns in addition to their loads a step that a lot of designers forget or evade during their practice.

Commonly several approaches are adopted as a simplification to solve the leaning columns problem or effective length in general like: Modified Nomograph Equation (Galambos, 1968), The Yura Approach (Yura, 1971), Lim & McNamara Approach (Lim & McNamar,1972), LeMessurier Approach (LeMessurier, 1977), Commentary Equations (AISC,1999). (L.Geschwinder 2002) , but the Direct Analysis Method is the most effective and accurate especially using the tools and software available in these days(AISC Design guide 28).

1.2.3 Performance based design:

The old designing philosophy of the earthquake resisting system was based upon achieving strength and serviceability limits were at most extend focusing on base shear and stories drifts and in case of more important structures the designer would multiply with some factors that only lead to increase in base shear and drifts and try to counter these additional amount of forces by increasing element sizes, this way of dealing with earthquakes led to catastrophic results in many events where failure mechanism were not accounted for , especially the plastic hinges development and energy dissipation in the structure and the redistribution of the forces and loads after hinges formation or failure of some elements, so at first it was developed the theory of weak beam strong column and several other arrangements were adopted in the codes to try to enforce a failure mechanism on the structure but at some level these procedures couldn't track the whole performance of the structure and the effect of redistribution of forces especially under such fuzzy effect as earthquakes since these changes or failure in system will lead to change in the period and in other word the response toward earthquake, at the same time the earthquake contain large spectrum (diverse) of frequencies and signals in both horizontal and vertical.

So, the commonly used designing spectrum which is based on (intensity design approach) was never enough.

Furthermore, the nature of the damage that expected or allowed was never effectively represented or reflected explicitly in codes, where codes in general allows alternative design method if it can be shown the structure behavior is equal or better than one designed according to the code (typically if according to peer reviews).

And here came the importance of performance design method where the elements are exposed to a selected series of real earthquakes acceleration which is of the same site conditions and scaled to match the code defined earthquake (Maximum considered earthquake MCE) , its behavior is captured through direct integrate methods that consider the most accurate way to stimulate the behavior throughout the earthquake which is for sure a nonlinear behavior at MCE , and the damage is assessed according objective or design goals (performance intended) which is represented by the mean of plastic deformation limits associated with amount of repair needed in each performance criteria.

This also mean that plastic design of the elements should be considered after normal elastic design, and plastic hinges should be modeled according to expected failure mechanism and the analysis result will display if the designer is successfully met the predefined design criteria and whether the failure mechanism is correct to assure the correctness of the modeling.

So, we will explain each step needed in the application:

1.2.3.1 Ground motion:

Commonly earthquake is believed to be result of tectonic plates movement deep underneath the earth surface, this theory consider earth to be divided into several plates as shown in Figure 1.26 that is solid and somehow floating on liquid or melted rocks and its movement is fueled by the earth rotation and since they are a separated plates their borders or lines between them are called fault lines and these fault lines mainly derived using the historical volcanic and earthquakes activity records, in other words these lines define where the largest and most powerful earthquakes tend to happen since what is believed to be happening on the event of earthquake is that two tectonic plates are colliding or moving in the opposite directions and as a result of this erupt or fracture large amount of energy is released as a propagating waves and these waves cause shaking to the earth surface and the structures above it.

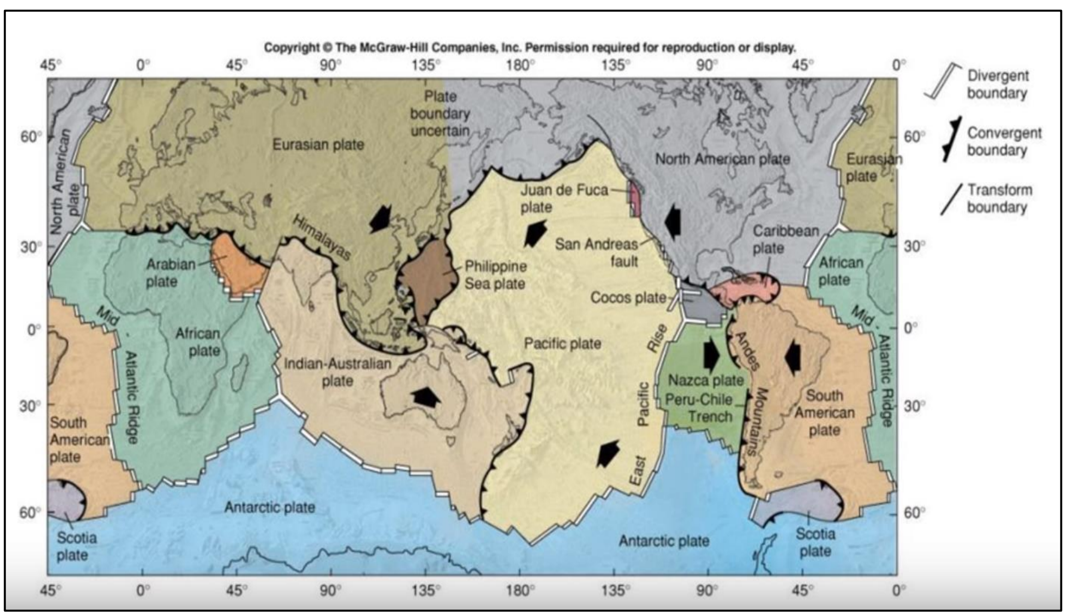


Figure 1.26: Tectonic plates (K. Franke 2017).

These waves (or earthquakes) are recorded as acceleration using accelerometers and we can derive ground motion velocity and displacement by integration as shown in Figure 1.27.

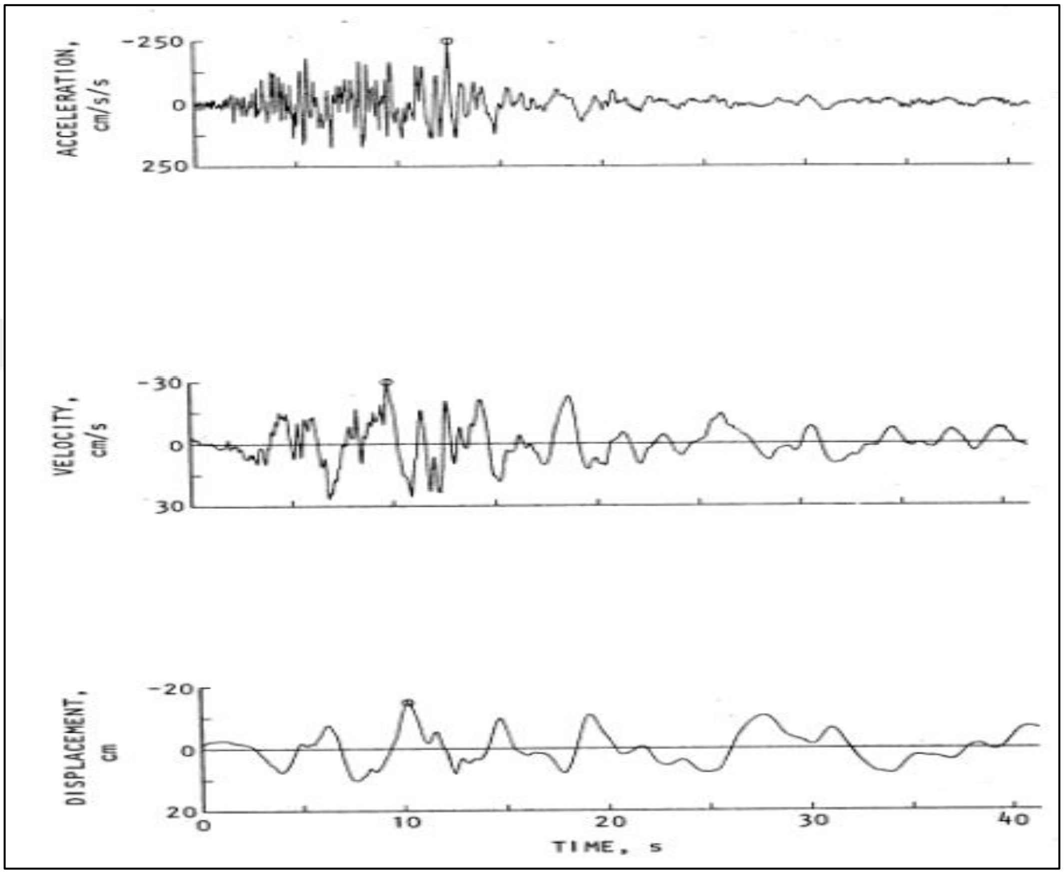


Figure 1.27: Ground motions records (FEMA P-1051).

In general, the forces or response of a structure due to earthquake is assessed using one of the three following methods:

Intensity-based assessment:

The main method adopted in the codes, relied on the 5% damped elastic acceleration response spectrum and obtaining the maximum intensity (response) at period equal to the structure's period (with consideration of site and soil properties).

Scenario-based assessment:

This method analysis the response of the structure due to earthquake event, by applying acceleration record of that event on the building (the computational model) and considering the forces and responses in the elements of the structure.

Of course, selecting and scaling process should be applied accounting to site and soil of the structure.

Risk-based assessment:

This method assesses the performance on designated (or selected) period of time like (1,25,50or100) years this mean it includes multiple intensities expected to happen during this period and then combined to predict the hazard that may accumulated over that period and then integrated with ground motion hazard curve to predict the annual rates of exceedance.

1.2.3.2 Selecting of suitable ground motions:

The main criteria of selecting of ground motions are magnitude, type of the fault, distance from the fault, and mechanism (site classification), shear wave velocity (referring to soil properties) and duration.

These properties should be similar or close to the target ground motion elastic response spectrum (that could be obtained from code as maximum considered earthquake).

In case of there is not enough records an Artificial or Synthetic Accelerograms can be generated , these methods were generally used because of the limited amount of records available or shared but one of the best tools that became available and solved this problem and also made selecting process easy is (Peer NGAwest Database) which can give access to a huge number of records from around the globe and allow easy to use search engine and even scaling of the selected records.

With the ability of to either use a record, event, code derived spectrum or user defined spectrum as a target. also, all other Important features like magnitude and site properties.

Caution should be taken in reviewing the result so that each earthquake is represented once, because sometimes it presents several records from different locations for the same earthquake.

1.2.3.3 Scaling of the ground motion records:

Scaling at period of interest:

According to this method the records are scaled at the natural period of the structure so that the response at this point is equal at both target and scaled spectrums, using the following scaling factor:

$$f = \frac{SA^{target}(T_n)}{SA^{record}(T_n)} \quad \textbf{(1.13)}$$

Scaling to a period range of interest:

According to this method records spectrums are scaled by average of relative values (scaling factor at each interval) over a period of interest often taken as $0.2T_n$ to $1.5T_n$ or $2T_n$ depending on the nature of structure (T_n: is the natural period of the structure) using one of the following expressions:

$$f = \frac{avg(SA^{target}(T_i))_{0.2Tn}^{1.5Tn}}{avg(SA^{record}(T_i))_{0.2Tn}^{1.5Tn}} \quad \textbf{(1.14)}$$

Or:

$$f = avg(\frac{SA^{target}(T_i)}{SA^{record}(T_i)})_{0.2Tn}^{1.5Tn} \quad \textbf{(1.15)}$$

Or: (Y.M. Fahjan 2008)

$$f = \frac{\sum_{T=T_A}^{T_B}(S_a^{actual} - S_a^{target})}{\sum_{T=T_A}^{T_B}(S_a^{actual})^2} \quad \textbf{(1.16)}$$

Or:

$$\ln f = \frac{\sum_i \; w(T_i) * \ln(^{SA^{target}(T_i)}\big/_{SA^{record}(T_i)})}{\sum_i \; w(T_i)} \quad \textbf{(1.17)}$$

$w(T_i)$: Weight of the value

To minimize the squared errors: (Kalkan and Chopra, 2010)

$$f = \frac{\sum_{i=1}^{n}(\bar{A}_i * \bar{A}_i)}{\sum_{i=1}^{n}(\bar{A}_i * A_i)} \quad \textbf{(1.18)}$$

Differences between 1st and 2nd is about 0.01, between 1st and 5th about 0.005 while 3rd and 4th give unreasonable results.

Spectral matching:

Another method used instead of scaling is spectral matching either in time domain or frequency domain, both of which is include some manipulation or changing of the content of the frequencies so it may change the nature of the record, it has several methods for applying but it is not used in this thesis since it may not represent the real effect the record should have on structure in my opinion, as demonstrated in Figure 1.28.

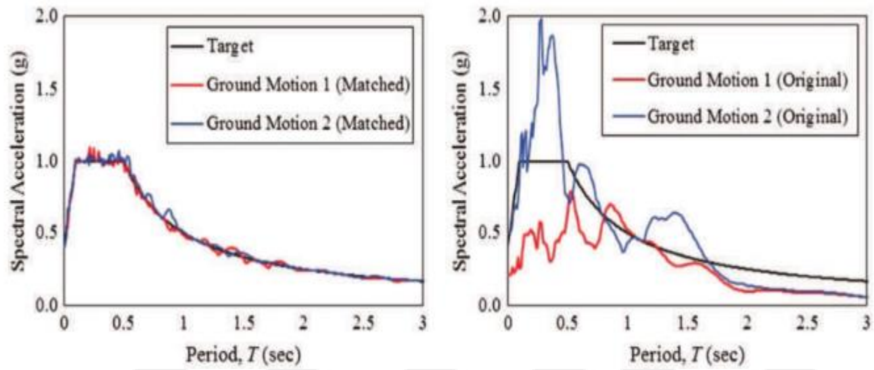


Figure1.28: Spectral matching.

1.2.3.4 Design criteria:

Typically, performance-based design is divided into four categories:

1-Immediate Occupancy (IO): where the cracks or plastic deformations due to seismic event are very small that they don't affect the integrity of the structure and the structure can continue to be used after the event without any need for repairs or a limited amount of repair.

2- Life Safety (LS): where the structure undergoes some damage and need to be assessed for future repair (in some cases structure need repair or some strengthening before being reoccupied) or use but the occupants receive few injuries during the event (typically LS is 75% of collapse presentation).

3-Collapse Prevention (CP): the structure undergoes huge deformations and cracks with the occupants exposed to high probability of being injured, and the structure repair is not efficient or can be considered complete economic loss.

There are some other performance levels (like operational, damage control, limited safety) but generally these three are the considered ones.

1.2.4 Direct time-integration methods:

Direct integration of the dynamic equilibrium equations of the structural system is considered the most comprehensive approach, generally these methods are divided into explicit and implicit methods.

Explicit methods uses differential equation at time (t) to predict the solution at (t+Δ), while implicit methods attempt to satisfy the differential equation at time (t) after the solution at time (t-Δt) has been found, implicit can use larger steps but they can be conditionally stable.

Many methods have been developed but in most cases these methods are assuming the function is smooth while the acceleration records are not, and due to the nonlinearity of the structural behavior (especially due to the effect of hysteresis of structural materials, buckling) some errors occur or the solution became unstable.

Newmark methods: in 1959, Newmark developed a one-step integration method to solve the dynamic problems like blasts and seismic loads, his method properties were changing according to values of (γ) and (β) to determine the stability of the solution, later on further improvements was introduced by other researchers to increase its accuracy and stability of the method, in table 1.7 family of Newmark's method (J.C. Golinval 2016).

Table 1.7: Family of Newmark's method

Algorithm	γ	β	Stability limit ωh	Amplitude error $\rho - 1$	Periodicity error $\frac{\Delta T}{T}$
Purely explicit	0	0	0	$\frac{\omega^2 h^2}{4}$	$-$
Central difference	$\frac{1}{2}$	0	2	0	$-\frac{\omega^2 h^2}{24}$
Fox & Goodwin	$\frac{1}{2}$	$\frac{1}{12}$	2.45	0	$O(h^3)$
Linear acceleration	$\frac{1}{2}$	$\frac{1}{6}$	3.46	0	$\frac{\omega^2 h^2}{24}$
Average constant acceleration	$\frac{1}{2}$	$\frac{1}{4}$	∞	0	$\frac{\omega^2 h^2}{12}$
Average constant acceleration (modified)	$\frac{1}{2}+\alpha$	$\frac{(1+\alpha)^2}{4}$	∞	$-\alpha \frac{\omega^2 h^2}{2}$	$\frac{\omega^2 h^2}{12}$

Another important modification to Newmark method is Hilber-Hughes-Taylor's method (HHT) which introduced a parameter (α) and the values of (γ) and (β) are determined according to (α) as follow:

$$-\frac{1}{3} \leq \alpha \leq 0 \qquad \textbf{(1.19)}$$

$$\beta = \frac{(1-\alpha)^2}{4} \qquad \textbf{(1.20)}$$

$$\gamma = \frac{1}{2-\alpha} \qquad \textbf{(1.21)}$$

In some sources (α) is defined as numerical dampening, since it can damp out the high frequencies which need very tiny time step to be considered and cause problem during solving as will be presented in the study, when $\alpha = 0$ the method is equivalent to Newmark method (average acceleration method) with $\gamma = 0.5$ and $\beta = 0.25$ and this gives the most accurate solution but in practical application it can produce excessive vibrations.

When $\alpha = -\frac{1}{3}$ it removes noise from period up to about 10 times the time step but can result in an inaccurate solution.

So, a value should be chosen suitable for the structure between the two values and should be as close as possible to 0 to increase the accuracy and generally this is done by trial.

And since HHT is recommended in the programs as main direct integration method it will be applied with different values of (α) and we will comment on the behavior of the solution.

2. APPLICATION

The model structure is 90x62.1 m one story industrial building with 2 spans (12.6m and 49.5m) and the small one contains a crane with lifting capacity of 20 tons, distance between frames is equal to 6 m , the wall height is 9 m and roofs are 10.2 m and 13.9 m. The project is located in Başakşehir area in istanbul with (D)soil classification, considered under wind loads of 90 km/hour and area with low vegetation.

The target design performance under MCE is IO, and the lateral force resisting system in frame direction is special moment frame and in the other direction concentric braced frame.

The computational model shown in Figure 2.1 is the final situation of the model also it should be noted that essential model was modified to achieve intended performance and to account for plastic design of the elements.

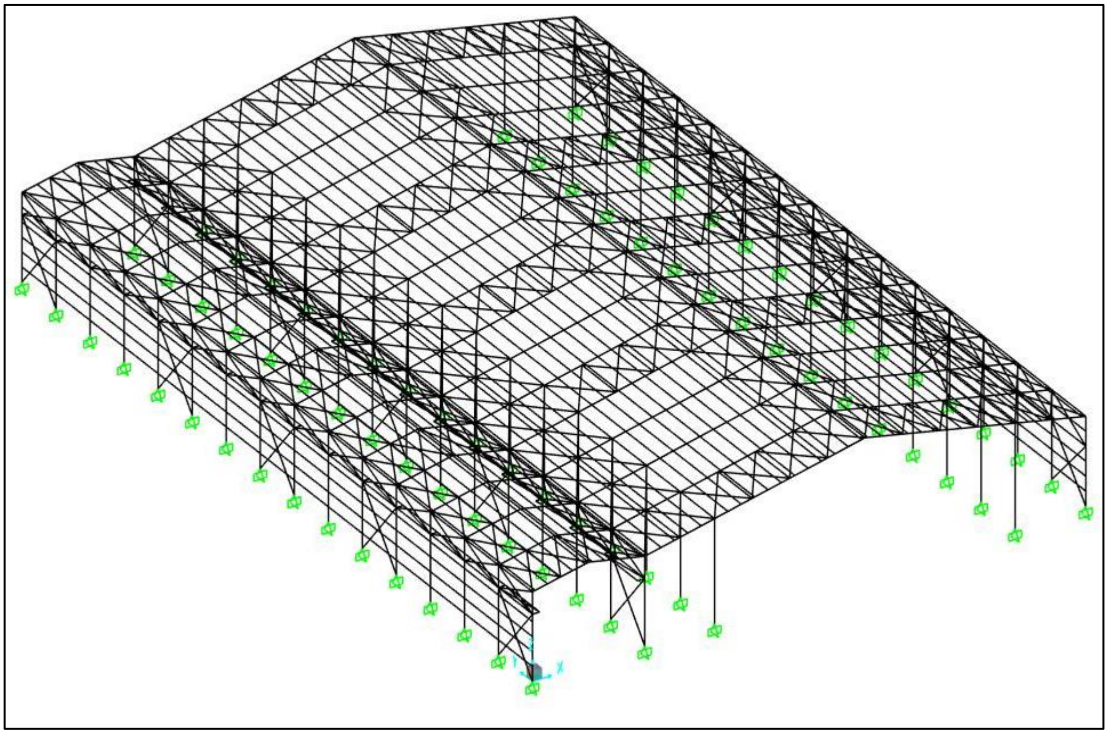


Figure 2.1: Final model.

2.1 Programs Used

SAP2000 to develop the model, analyzing and applying direct integration.

SEISMOSIGNAL to analyze the records and obtain the spectrum for each signal.

Excel & **Smath** for side calculation including (scaling of signals, wind load distributions, design of elements and connections……).

IDEASTATICA to analyze and design some connection using Finite elements method.

AUTOCAD to draw some necessary figures.

Tekla Structures to develop as construct BIM model of the structure with its elements and connections.

Robot Structural analysis to simulate crane loads and determine most critical arrangements.

2.2 Material used

Structural steel: as shown in Table 2.1.

Table 2.1: Properties of the used structural steel

	Fy (MPa)	Fu (MPa)	E (MPa)	γ (ton/m^3)	Ry
S275	275	430	210000	7.849	1.1
S355	355	510	210000	7.849	1.1

Welding material: Tempo B 60 Fe=580 MPa or E80

Bolts: A490(10.9)   Fyb=900MPa, Fub=1000 MPa

2.3 Loads:

2.3.1 Dead loads

Dead load includes the weight of the covering panels for roof and walls equal to (0.1kN/m^2) also self-weight of the elements considering by taking the self-weight multiplier as 1 in Dead load definition in Sap2000 to consider it automatically

2.3.2 Live loads

Live loads are considered for repair of roof panels equals to (0.5 kN/m^2)

2.3.3 Snow loads

According to TR EN 1991-1-3 snow load is given as follow:

$$S = U_i * C_e * C_t * S_k \quad \textbf{(2.1)}$$

Istanbul is categorized in II area with height less than 500 m so $S_k = 0.75kN/m^2$

According to the site and situation $C_e = C_t = 1$

Table 2.2: Snow load shape coefficients (EN 1991-1-3).

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$60° \leq \alpha$
μ1	0.8	0.8 (60-α)/30	0
μ2	0.8+0.8 α/30	1.6	------

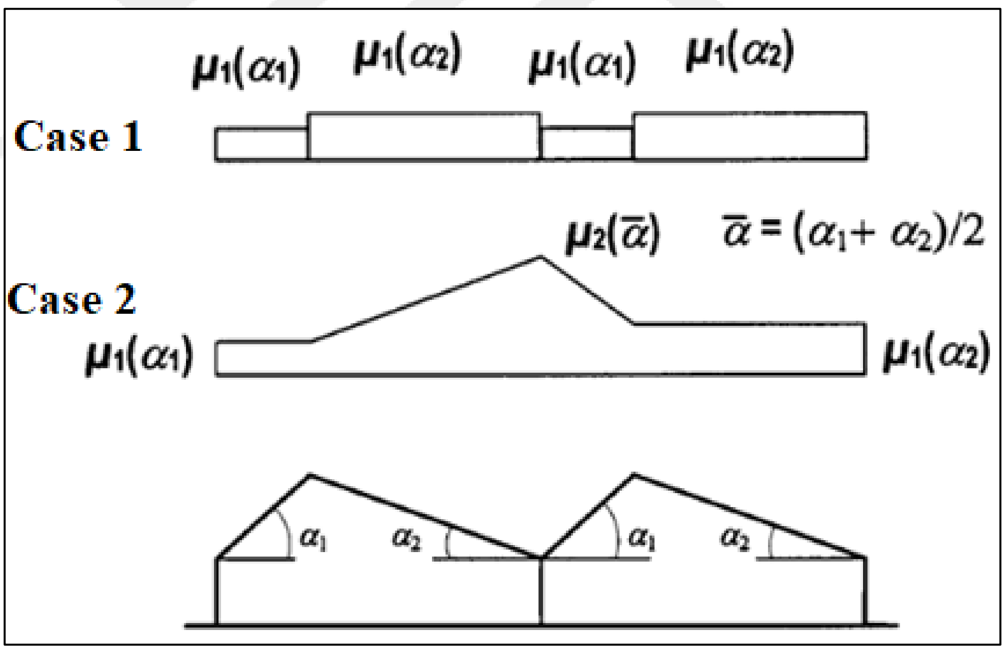


Figure 2.2: Snow load shape coefficient on multi-span roofs (EN 1991-1-3).

Case 1: Snow loads are considered without any accumulation.

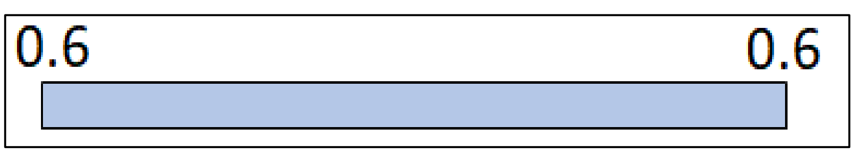


Figure 2.3: Snow load arrangement in case 1.

Case 2: Snow accumulated between the roofs.

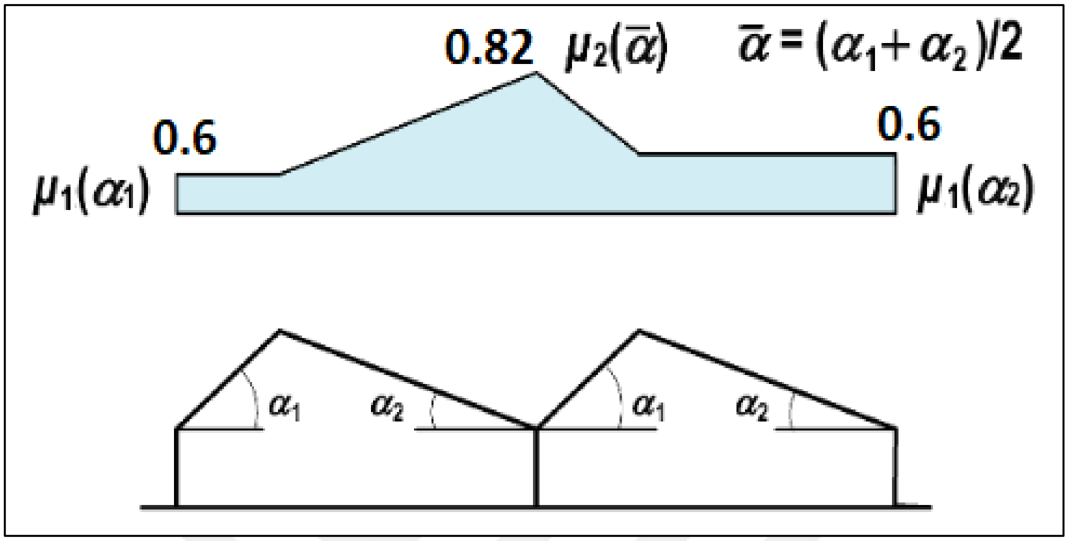


Figure 2.4: Snow load arrangement in case 2.

Snow load shall be applied in vertical direction (gravity direction) on purlins according to the distance between them.

2.3.4 Wind load

Due to differences between to spans of the building there will be 3 loading cases for wind directions 0°, 90° and 180°, According to TR EN 1991-1-4.

The characteristic wind speed is assumed $V_{b,0} = 90 \frac{km}{hour} = 25 \frac{m}{sec}$, and terrain category (II), $Z_0 = 0.05\ m$, $Z_{min} = 2\ m$.

$V_{b,0}$: “The characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights”. (EN 1991-1-4).

$$V_b = C_{dir} * C_{season} * V_{b,0} = 25 \frac{m}{sec};\ C_{dir} = C_{season} = 1 \qquad \textbf{(2.2)}$$

Velocity variation with height:

$$V_m(z) = C_r(z) * C_0(z) * V_b \qquad \textbf{(2.3)}$$

Terrain roughness:

$$C_r(z) = K_r * \ln(\frac{z}{z_0}) \quad \textbf{(2.4)}$$

$$K_r = 0.19 * \left(\frac{Z_0}{Z_0, II}\right)^{0.07} = 0.19 \quad \textbf{(2.5)}$$

$C_0(z) = 1$, orography factor.

Standard deviation of the turbulence:

$$\sigma_v = K_r * V_b * K_I = 4.75 \; ; \; K_I = 1 \; turbulence\ factor \quad \textbf{(2.6)}$$

Turbulence intensity:

$$I_v(z) = \frac{\sigma_v}{V_m(z)} \quad \textbf{(2.7)}$$

Peak velocity pressure:

$$q_p(z) = [1 + 7 * I_v(z)] * \frac{1}{2} * \rho * V^2{}_m(z) = C_e(z) * q_b \quad \textbf{(2.8)}$$

$\rho = 1.25\ kg/m^3, air\ density.$

$$q_b = \frac{1}{2} * \rho * V^2{}_b \quad \textbf{(2.9)}$$

Table 2.3: Wind pressure at reference height.

Heights	Cr(z)	Vm(z)	Iv(z)	$q_p(z)$ (kg/m^2)
9	0.99	24.66	0.193	892.88
10.2	1.01	25.25	0.188	923.44
13.9	1.07	26.72	0.178	1001.66

Wind pressure at surface:

$$w = q(z_0) * (Cpe - Cpi) \quad \textbf{(2.10)}$$

We considered the area of the opening at dominant face is twice the area of the opening in the remaining faces so:

$$C_{pi} = 0.75 * C_{pe} \quad \textbf{(2.11)}$$

The pressure due to wind effect is the difference of the resultant interior and exterior pressure according to their direction (either pressure (positive) or suction(negative)) and the most unfavorable arrangement will be considered as shown in Figure2.5.

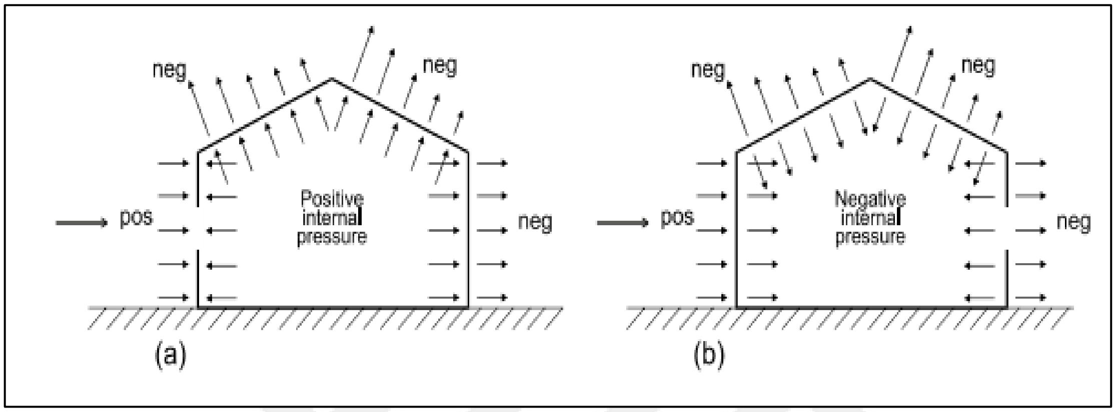


Figure 2.5: Internal and external pressure arrangements. (EN 1991-1-4).

$C_s, C_d = 1$, Structural factors and equal to (1) in structures with height less than (15) m.

C_{pe} Calculation

These calculations is a tricky and we will consider them and was considered for each direction as separate case but here we will mention the main arrangements that was followed.

For the roof

In multi-span roofs the C_{pe} is taken according to the Figure 2.6.

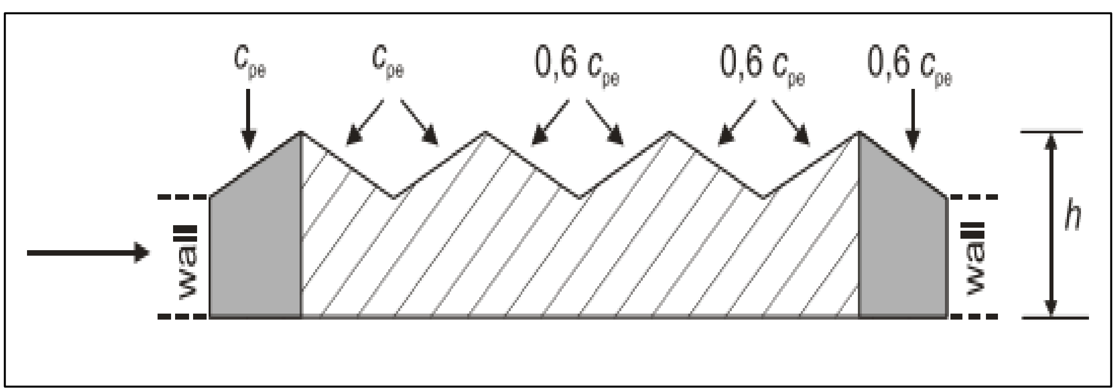


Figure 2.6: External pressure coefficient on multi-span roof. (EN 1991-1-4).

Also, it should be noted that according to the code "the first C_{pe} is the C_{pe} of the monopitch roof (Figure 2.7), the second and all following C_{pe} should be taken from duopitch roof (Figure 2.8).

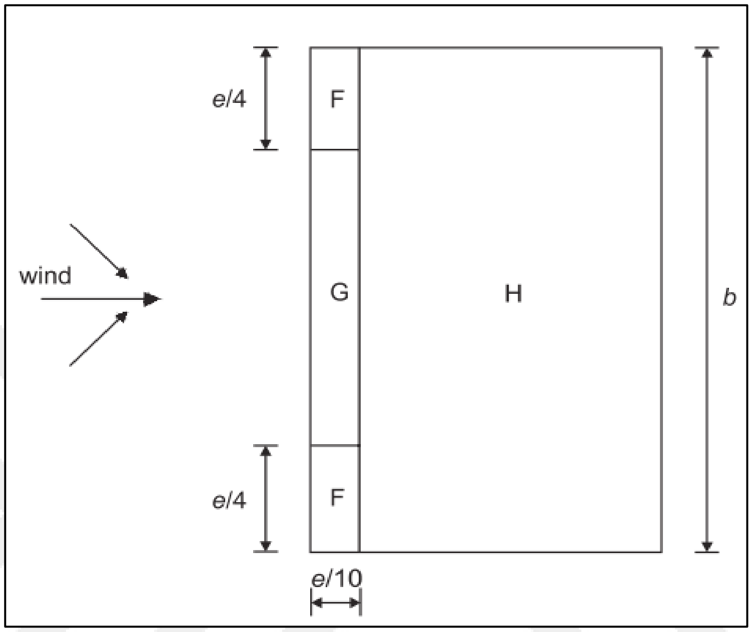


Figure 2.7: C_{pe} Coefficients distribution on monopitch roof (EN 1991-1-4).

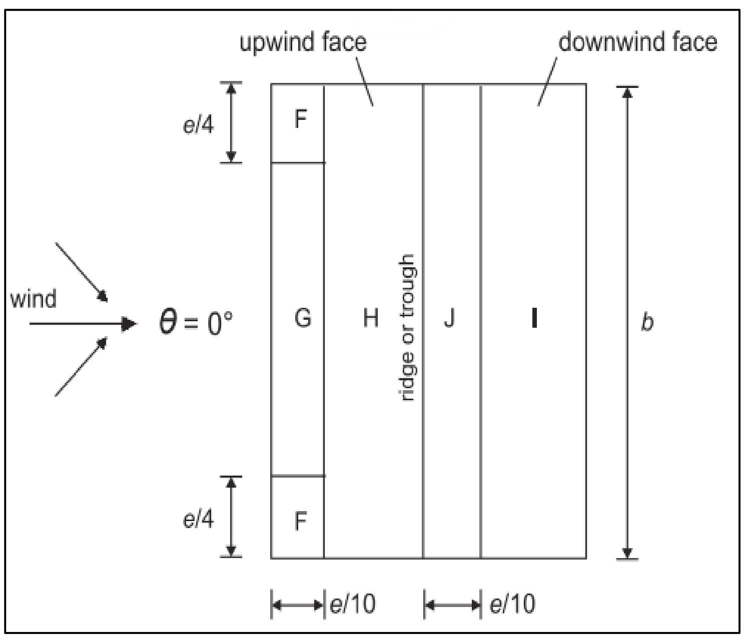


Figure 2.8: C_{pe} Coefficients distribution on duopitch roof (EN 1991-1-4).

Where $e = \min(b, 2 * h)$; b *is crosswind dimension.*

For 90° the arrangement for each span as shown in Figure 2.9.

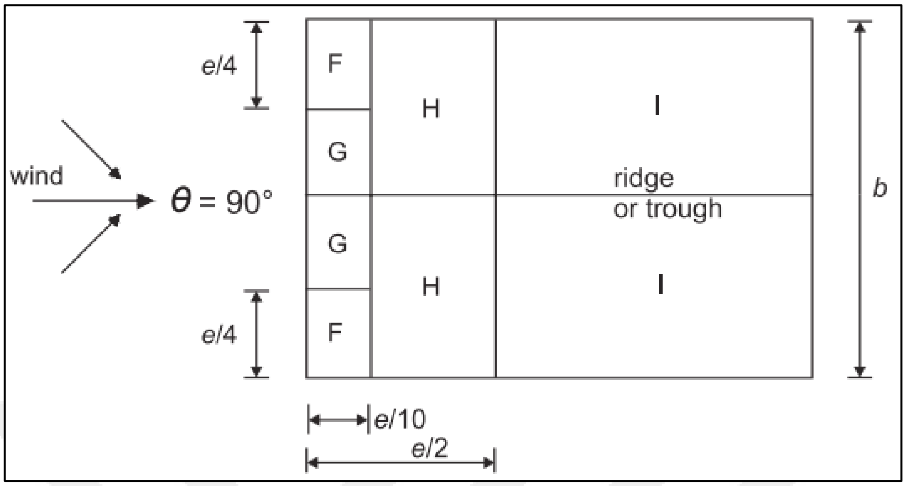


Figure 2.9: C_{pe} Coefficients distribution on each span roof (EN 1991-1-4).

Also, "the zones F/G/J used should be considered only for the upwind face, the zones H and I should be considered for each span of the multi-span roof". (EN 1991-1-4).

The roof C_{pe} values are presented in the Table 2.4.

Table 2.4: C_{pe} Values for roof according to direction of the wind

	F	G	H	I	J
0°	-0.621	-0.546	-0.173	-0.299	-0.746
90°	-1.414	-1.3	-0.638	-0.538	-
180°	-0.621	-0.546	-0.173	-0.299	-0.746

For walls: The arrangement is shown in Figure 2.10, and the C_{pe} values are presented in Table 2.5.

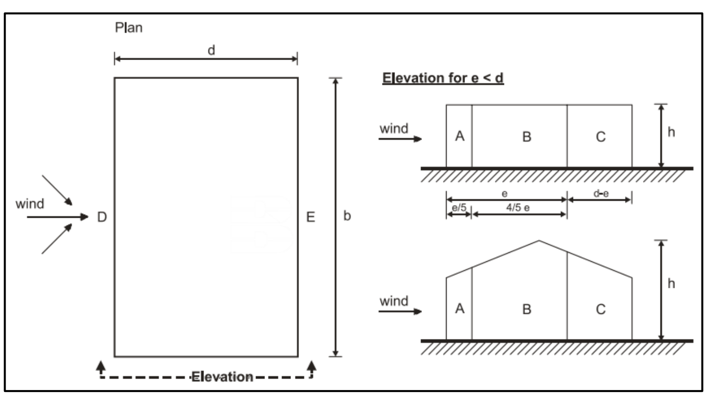


Figure 2.10: C_{pe} coefficients distribution on wall

Table 2.5: C_{pe} Values for wall in all directions

A	B	C	D	E
-1.2	-0.8	-0.5	0.7	-0.3

The wind load was applied to the wall rails and purlins according to three wind directions and considering different values of internal and external pressure coefficients.

Also, in front and back sides the doors potions were considered and the loads gathered by wind columns was applied according to each case directly to their connect point in the frames (wind columns were not modeled).

2.3.5 Crane loads

Crane properties was taken from (stlah im hochbau) with lifting capacity (20) tons and spans of (11.25) m with wheel distance equal to (3.84) m and the maximum wheel load was equal to (12.5) tons (it was mentioned in the book a difference between front and rear wheel but this is not the current assumption in the practice).

In addition to vertical impact loads, crane produce lateral forces of (20%) of the vertical load (except bridge weight) and longitudinal forces equal to (10%).

A computer simulation was run on (Robot structural analysis) to determine the most critical position of the wheel for both runway beams and on its supporting cantilevers.

For fatigue it was considered that crane have 10 passes per hour 365 days for 50 years, also, it was compared with other method of amplification the load to account for fatigue which was found to be more conservative.

Runway beam reactions was applied on the structure on columns cantilever and considered as live load in load combinations.

Runway beam design (loads and fatigue design) for two methods and load arrangements are presented in the Appendix D.

2.3.6 Seismic loads

Using the website provided by “Afet ve Acil Durum Yonetimi Baskanligi”, https://tdth.afad.gov.tr by selecting the coordinates of latitude :41.08989 & longitude: 28.789307 Figure2.11, soil type: D, spectrums were obtained of the service level earthquake (DD-3 50% probability of occurring in 50 years or 72-year repeat occurrence)

Figure2.12, design level earthquake (DD-2 10% probability of occurring in 50 years or 475-year repeat occurrence) Figure2.13, and the maximum considered earthquake (DD-1 2% probability of occurring in 50 years or 2475-year repeat occurrence) Figure2.14.

The detailed report for each is attached in the Appendix C.

Figure2.11: Supposed location of the building.

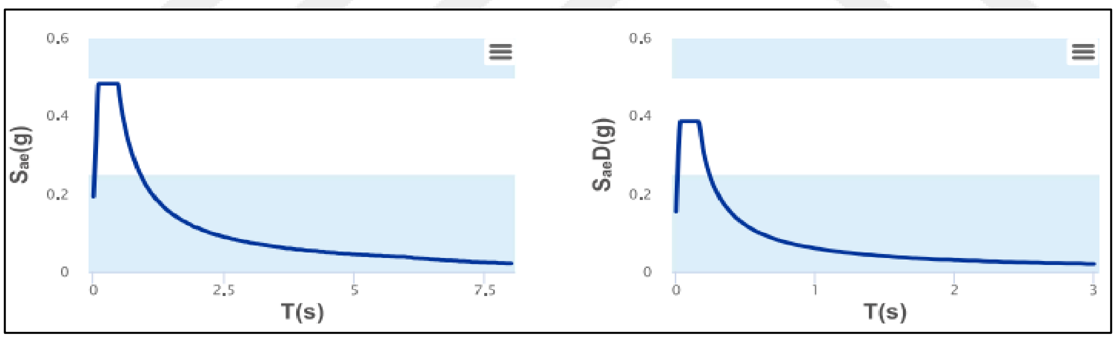


Figure2.12: DD-3 Accelerations spectrums. (Horizontal “left”, Vertical “right”).

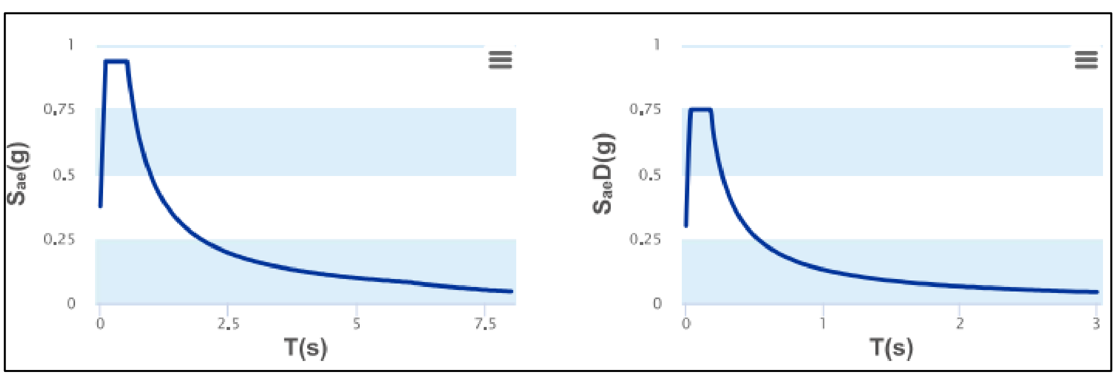


Figure2.13: DD-2 Accelerations spectrums. (Horizontal “left”, Vertical “right”).

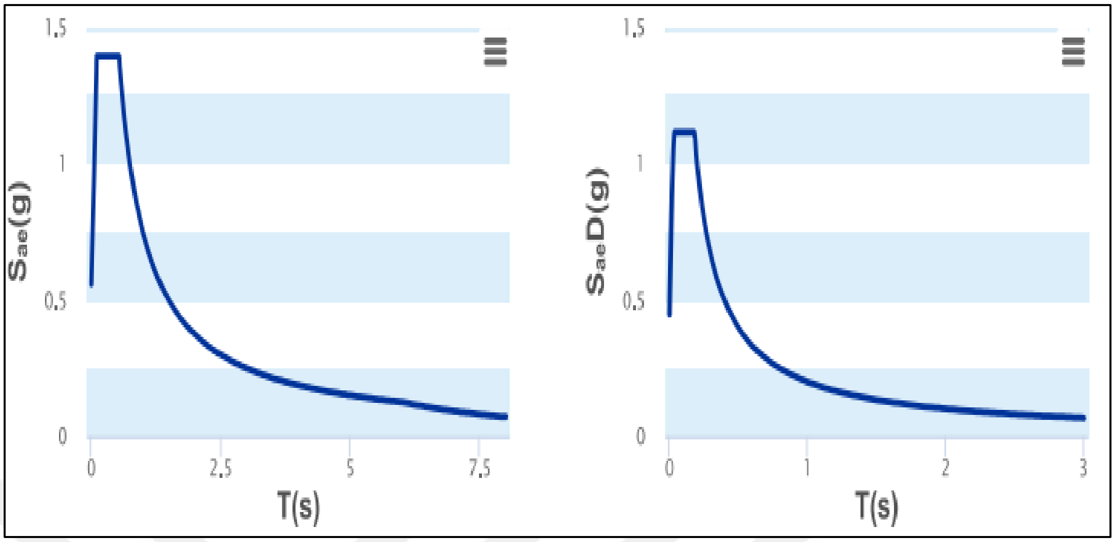


Figure2.14: DD-1 Accelerations spectrums. (Horizontal “left”, Vertical “right”).

DD3 was used to test the integrity of structure, DD2 was used for sizing and designing of the elements DD1 was used for scaling the earthquake signal for performance analyzing.

In the structure the X direction is considered as steel special moment frame with

$R = 8 , \Omega_0 = 3 , C_d = 5.5 , I_e = 1.5.$

While, In the Y direction is considered as steel special concentrically braced frames with $R = 6 , \Omega_0 = 2 , C_d = 5 , I_e = 1.5.$

Mass sources: dead load, crane operational weight (as a permeant equipment).

Note: snow load was ignored since the flat roof snow load is less than 1.44 kN/m^2.

2.3.7 Scaling of acceleration records

A group of 7 suitable orthogonal pairs was chosen after obtaining their spectrums using SEISMOSIGNAL they were scaled using 5th suggested equation to match the MCE (DD1) over the range 0.2Tn to 1.5Tn as presented in Figure 2.15.

Signals records are reported in the Appendix A.

2.3.7.1 Kocaeli spectrum problem

Also, it should be noted that Kocaeli earthquake spectrum had a unique characteristic as presented in Figure 2.16, so it was not considered in the study (three records from two sources was obtained to avoid any mistake).

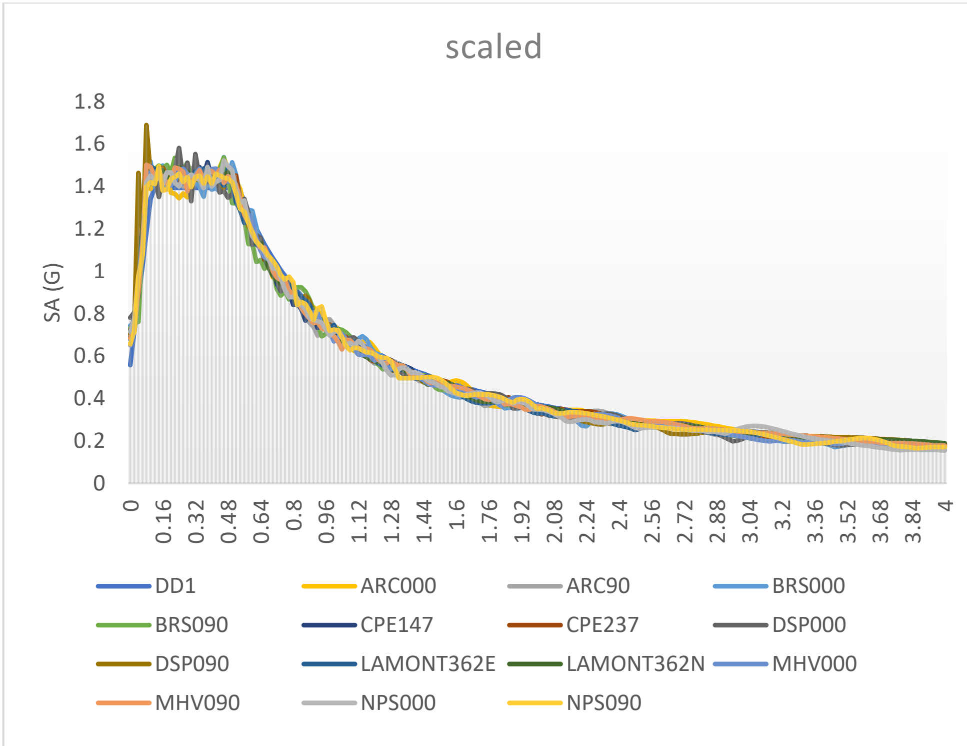


Figure 2.15: Spectrums scaling.

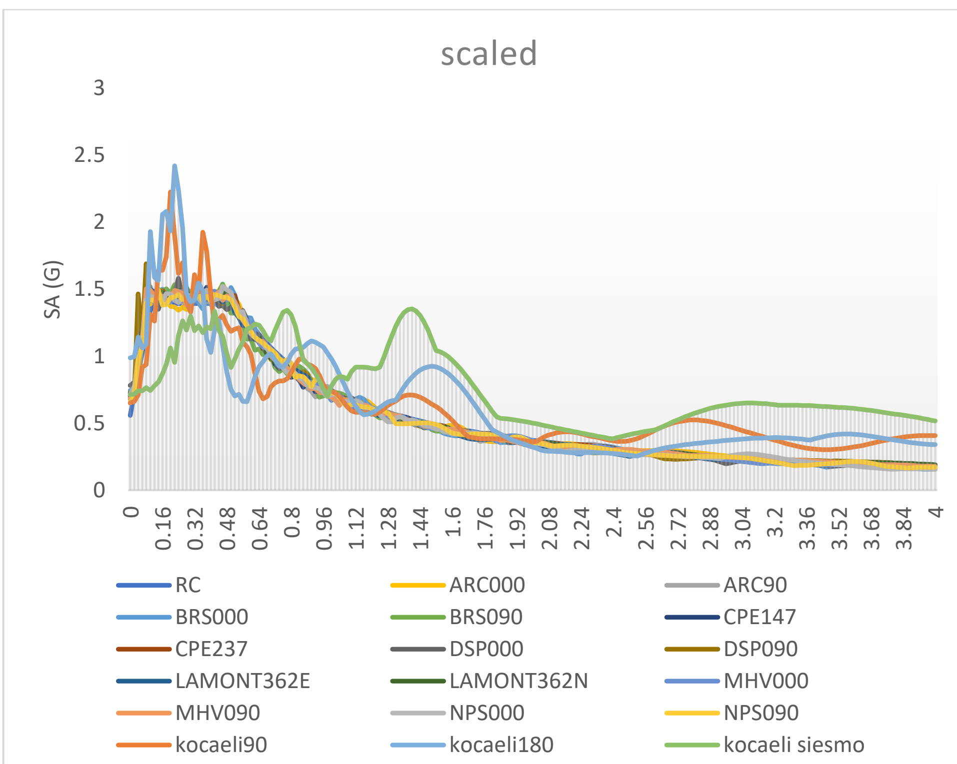


Figure 2.16: Scaled spectrums including Kocaeli earthquake records.

So, it was removed from the study because it may need further investigating and considerations (even in long periods it shows high response).

Note that due to changes applied to improve the structure performance, its natural period has changed a little but upon investigation scaling factor were parley changed to extent that can be ignored.

After modifying the signals records with the scaling factors, they were defined as time history records and used each two orthogonal pairs as a load case.

Direct integration was applied and a comparison between Newmark and Hilbe-Hughues-Taylor (with several values of α) was done and solution behavior was monitored and reported in the conclusions.

2.3.8 Notional loads:

A group of notional loads is defined and applied to the structure with consideration of direction and signal equals to 0.002 of the vertical loads.

2.4 Load Combinations

1) $1.4\,D$

2) $1.2D + 1.6C_i + 0.5S_j\ ; i = 1,2,3,4 \quad j = 1,2$

2) $1.2D + 1.6C_i + 0.5LR\ ; i = 1,2,3,4$

3) $1.2D + 1.6(LR) + C_i;\ i = 1,2,3,4$

3) $1.2D + 1.6(LR) + 0.5W_k;\ k = 0,90,180$

3) $1.2D + 1.6S_j + C_i;\ i = 1,2,3,4\ \ j = 1,2$

3) $1.2D + 1.6S_j + 0.5W_k;\ \ j = 1,2 \quad k = 0,90,180$

4) $1.2D + C_i + 0.5S_j + W_k\ ; i = 1,2,3,4\ \ j = 1,2 \quad k = 0,90,180$

4) $1.2D + C_i + 0.5LR + W_k\ ; i = 1,2,3,4\ \ k = 0,90,180$

5) $0.9D + W_k\ ; k = 0,90,180$

6) $1.2D + Ev + Eh + C_i + \ 0.2S_j\ ; i = 1,2,3,4 \quad j = 1,2$

7) $0.9D - Ev + Eh$

Where $Ev = 0.2S_{DS}D$ And $Eh = Ex + 0.3Ey\ or\ Ey + 0.3Ex$

Also, it should be noted that during Direct Analysis application these combinations will be transformed into nonlinear load cases after adding notional loads to capture the effect of P-Δ due to interactions between different forces that can't be reflected in linear summation.

The total number of load combinations was more than 200 combinations so we presented them in this way.

2.5 Steps to Implement the Direct Analysis Method (using Sap 2000)

1-Correctly model your structure (including modeling initial imperfection if notional loads won't be applied -preferably avoid modeling imperfection and apply notional loads).

2-Define the national loads for all gravity loads (simpler than applying geometrical imperfection).

3- Convert load combinations to nonlinear load cases with considering P-Δ to capture the interaction between the loads through second order analysis.

5-Add notional loads effects to the lateral loads' combinations (this is a simpler approach than considering $\frac{\Delta_{2nd}}{\Delta_{1st}}$ ratio).

4-Impose auto meshing on the elements not less than 4 parts.

5- Put analyzing method as direct analysis method and the τ_b is variable to be considered later.

5-Run the analysis for the first time.

6- Run the design process so that the reduction in stiffness is applied with the consideration of the value of τ_b being accounted and calculated through designing process.

7-Unlock the model and re-run the analysis again (this second time is the direct analysis) now the results are ready to be used in design.

3. DESIGN

After implementing direct analysis method, we could run a preliminary design in the program to identify the riskiest combinations and critical element of each group or we can have it from results tables

Elements were first designed and sized with normal design procedures for elastic element with consideration on limits of thickness according to special frames limits for highly seismic region and later on they were checked using plastic design philosophy to account for after-yielding behavior and forces redistributions and achieve targeted performance and assure the intended behavior

3.1 Gusset Plates Design

There are several methods mentioned in the literature to design gusset plates in the corner of the braces(like KISS Method, Parallel Force Method, Truss Analogy Method & Uniform Force Method) (AISC Design Guide 29)but there are some special considerations that should be taken regarding that these braces are part of special concentrically braced frame (341-16 section 7.2) states “Bolts and welds shall not be designed to share force in a joint or the same force component in a connection” which leave only KISS Method to be used since UFM violate this assumption as shown in Figure 3.1 & Figure 3.2.

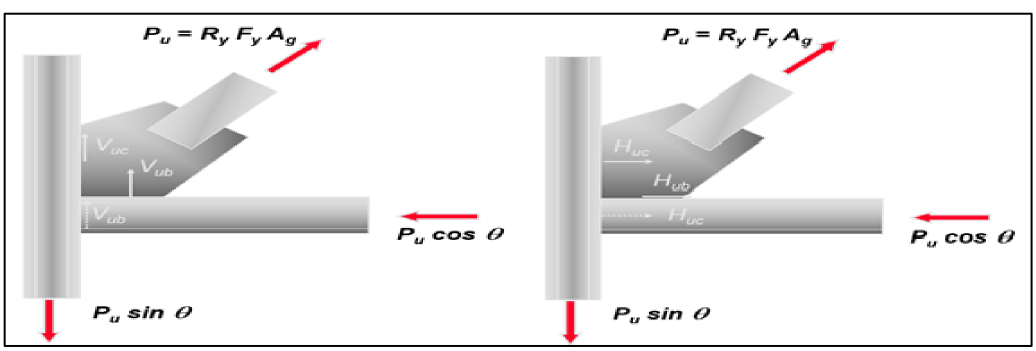


Figure 3.1: UFM Forces distribution assumption (B.Ö. Çağlayan, 2018).

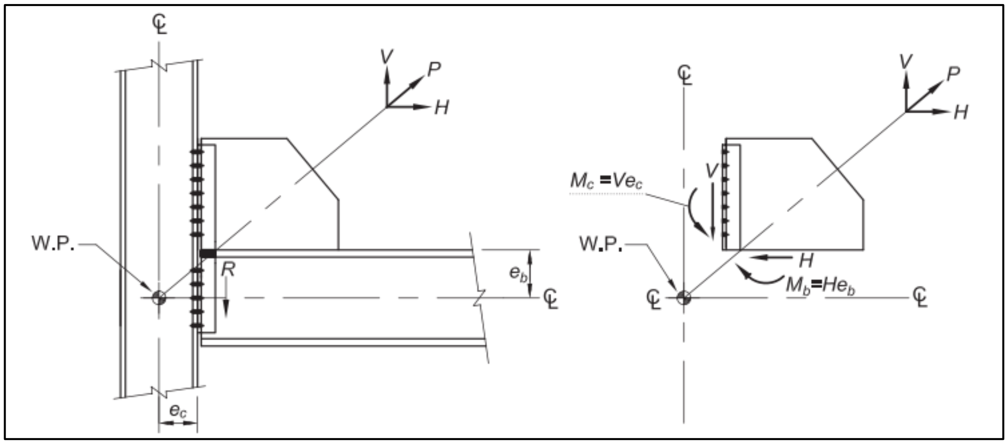


Figure 3.2: KISS Forces distribution assumption (AISC Design Guide 29).

Another important point in dimensioning the gusset plate is "fold line" should be equal to (2t) as shown in Figure 3.3 to secure pinned end braces (which mean flexural plastic hinge will form at middle as shown in Figures 3.4 & Figures 3.5) so the braces will not impose moments on connections and connected members.

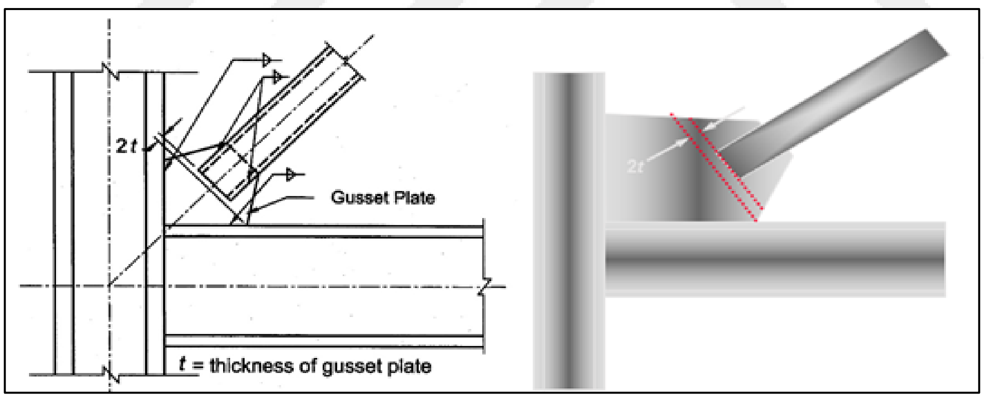
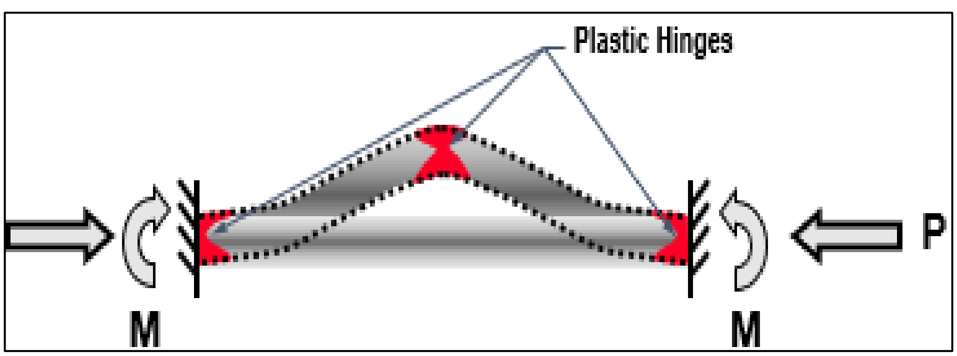
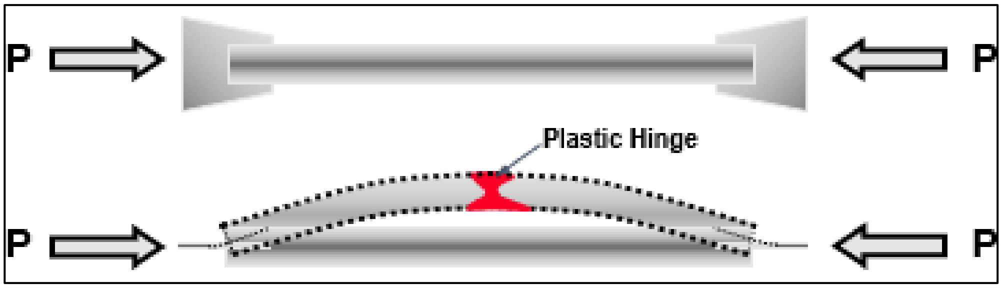


Figure 3.3: Fold line (B.Ö. Çağlayan, 2018).



Figures 3.4: Hinges development in fixed ends braces (B.Ö. Çağlayan, 2018).



Figures 3.5: Hinges development in pinned ends braces (B.Ö. Çağlayan, 2018).

The forces estimated to be imposed by braces on the connection are:

a) braces in tension:

$$P = Ry * Fy * Ag \quad \textbf{(3.1)}$$

b) braces in compression:

$$P_{max} = 1.1 * Ry * Fcr * Ag \quad \textbf{(3.2)}$$

$$\text{or } P_{residual} = 0.3 * Ag * Fcr \quad \textbf{(3.3)}$$

Also, the elements surrounding braces should resist the accumulated forces developed by the yielding multiple stories of braces as shown in Figure 3.6.

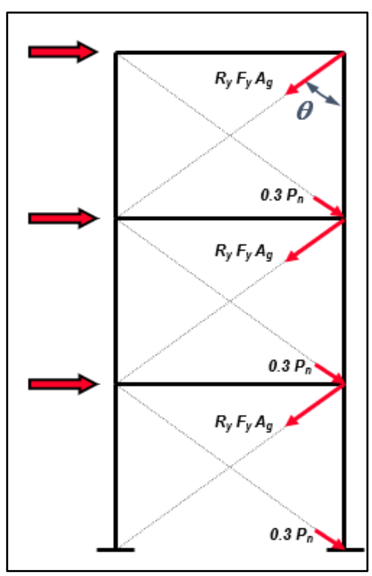


Figure 3.6: Forces need to be added and resisted by columns and beams (B.Ö. Çağlayan, 2018).

3.2 Base Plate Design

Base plate was designed following the (AISC Design Guide 1) for base plates and anchors and the complete design is demonstrated in the Appendix D.

But later on, we decided to model it on IDEASTATIKA which consider the finite element and exact behavior of the elements it was clear that current practice of putting bolts in a line doesn't reflect the reality that stresses and forces are concentrating in the middle bolts which leads to early failure of the bolts before reaching the supposed design forces as demonstrated in Figure 3.7.

So, curved arrangement was adopted which lead to equal distribution of the forces over bolts as shown in Figure 3.8

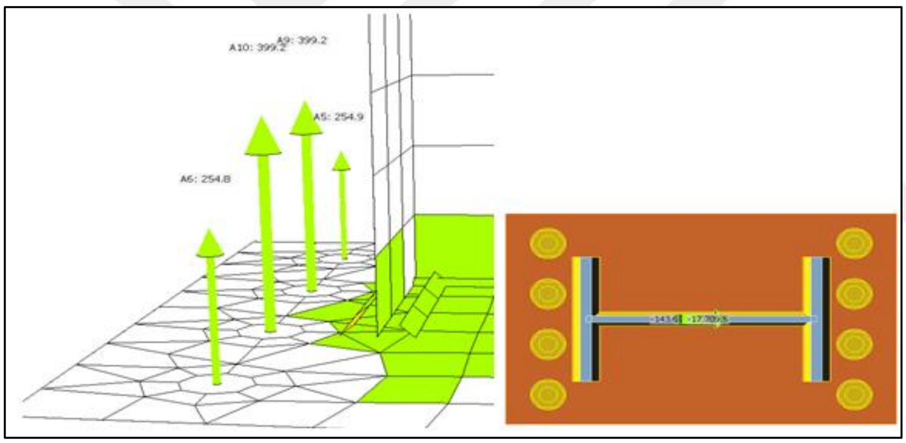


Figure 3.7: Forces distribution in line assumption.

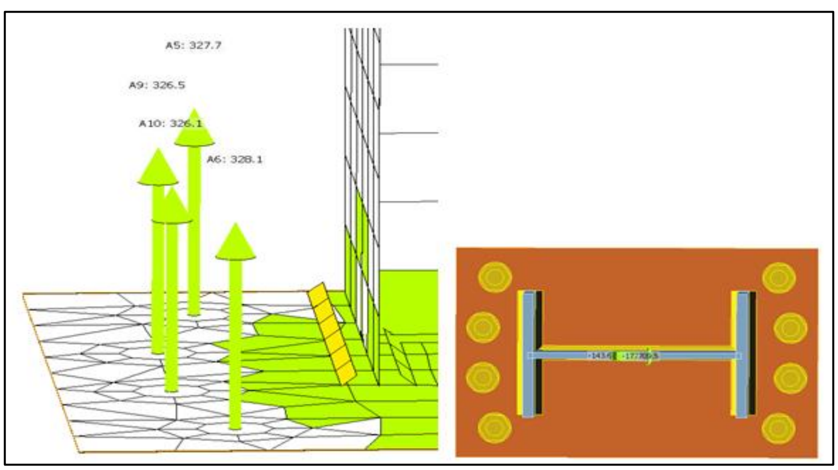


Figure 3.8: Forces distribution in curved arrangement.

In the presented arrangement middle bolts was moved 25mm but many other models were developed in search for an equation to define this modification or standardize it but we failed to obtain a consistent equation, but mainly it is related to section height and flange size.

3.3 Frames Connections (Prequalified Connections)

Rafter to column connections were designed as prequalified connections according to AISC 358-16 and the only suitable type according to the prequalification conditions was “welded unreinforced flange-welded web (WUF-W) moment connection”, and the design is presented in the Appendix D.

Apexes’ connections were designed in IDEASTATIKA.

3.4 Performance Based Design Steps:(using sap2000):

1- Develop accurate model for your structure and introduce lateral forces resisting system that satisfy the requirements in local codes and provides efficient and continuous load path.

2-Size and design element to satisfy all strength and serviceability requirement including lateral loads like seismic loads at design level (10% in 50 years).

3-Select a group of earthquake signals (orthogonal pairs) that share the same characteristic properties with structure site like magnitude and fault properties, preferred to be from local records but if not enough other foreign resources like PEER NGAwest Database or in some cases artificial or synthetic accelerogram records can be generated to achieve the required number of records pairs demanded by code.

4- Obtain spectrums of the selected signals.

5-Scale spectrums selected signals (each record alone) with MCE spectrum (2% in 50 years) with suitable scaling method (either period, range of periods or by matching).

6- Apply scaling factor to signal records and define the scaled signals as time-history function.

7-Assign the suitable hinges according to the supposed behavior of the elements.

8-Define load cases for each pair of the scaled records and choose the case to be (time history function, nonlinear and direct integration with suitable number of output times), and define a load case include the mass source to define the starting state for time history load cases.

9-Choose the suitable direct integration method (Newmark, HHT …) (it is recommended to use HHT method and to try several values of α to find the best fit for the project as will be explained later).

10-Run the analysis for one pair at a time, and monitor the behavior of the solution to assure the right choice of solving method.

11-In case the structure didn't met the expected performance review and modify the lateral forces resisting system and improve its integrity and re-run the analysis again for all cases to make sure the structure meets performance goals intended.

It should be noted that output steps and method of direct integration determination is very important and sensitive and can increase the size of the file a lot (in one analysis the file size after analyzing was more than 63 gigabytes).

3.4.1 Hinges definition

For the hinges we defined them according to the New Turkish seismic code (TSC,2017) Appendix-5D Tables 5D.1 and 5D.2 for the frames and 5D.4 for the braces, same values are provided as auto hinges under ASCE 41-13 code in SAP2000 package.

It should be noted that in the new Turkish seismic code is adopting Ry values similar to American code while considering using materials according to European material which have different Ry values (Ry=1.1).

4. RESULTS AND DISCUSSION

In this study we tried to highlight many topics related to steel structures especially industrial buildings and performance under seismic loads we will present the result we obtained considering main aspects:

4.1 Fatigue

In Fatigue using the current method of S-N charts is more reliable and economical compared to the old method of increasing the imposed load by specific factors. allowing the consideration of each case and the number of cycles per life time to be defined explicitly.

4.2 Direct Analysis Method

Direct analysis method is most accurate available method since it considers several effects missed or ignored by other methods and contain second order analysis to closely capture the behavior of the real structure as mentioned before.

On the other hand, there is no restrictions on using direct analysis method like other methods.

Engineers are highly encouraged to implement it in their practice benefiting from high analyzing power of computers and to avoid wrong estimations in the complex structures, and to provide more accurate yet economic solutions.

4.3 Wind Load

The procedure for considering wind loads currently provided in the codes is complex and time consuming to implement in practice, it is recommended to develop more suitable arrangement with less complexity yet can be adopt to various shapes of the roofs (in our situations I believe that the current arrangement is not suitable especially with inequal heights of the roof), or allow a simulations-based assessment for wind flow around the buildings which can provide accurate characteristic definition for the wind effect on the building, a computer programs should be developed to perform flow simulation which may serve this purpose more effectively, in fact this is being investigated through what is called (Database assisted wind load application) relaying on large database of wind tunnel tests records with some codes to find the suitable load for the building being designed, which is expected to be integrated in the future codes.

4.4 Performance Based Design

-In relation with performance-based design, we mentioned selecting of the earthquake signals to suit the intensity and site properties.

-When scaling the records and comparing between multiple methods for scaling and matching, it was found that the best method for scaling is to minimize the squared errors using the following factor: (Kalkan and Chopra, 2010).

$$f = \frac{\sum_{i=1}^{n}(\bar{A}_i * \bar{A}_i)}{\sum_{i=1}^{n}(\bar{A}_i * A_i)} \qquad \textbf{(4.1)}$$

While other methods mentioned provided either close values like 1st, 2nd or some errors like 3rd, 4th and applying spectral matching was avoided since it manipulates the frequency content.

4.4.1 HHT integration method and α values

α values ranges from 0 to -1/3 and when its value is (0) converts HHT into Newmark method (average constant accelerations "the most accurate method"), but as its value decreases away from (0) it starts to impose numerical damping of high frequency signals that need tiny step to consider in the integration according to the literature, several values was suggested in program manual (-1/12 ,-1/24), but was considered five values (0, -1/48,-1/24,-1/12,-1/3).

The solution behavior was monitored and it was found that:

For (**α=0**) the analysis is very slow and processing resources demanding and at each event point like hinges forming or forces redistributing the time step is becoming smaller and smaller with each iteration (in one analysis it reached around ($7*10^{-6}$)sec as shown in Figure 4.1) and it can be said it stuck and no longer advancing (in one analysis it barely advanced after certain point even we waited for a week) and at the end the analysis was terminated because of what can be described as the vibrations induced by these high frequencies caused some sort of disintegration to the model which led the program to end the process.

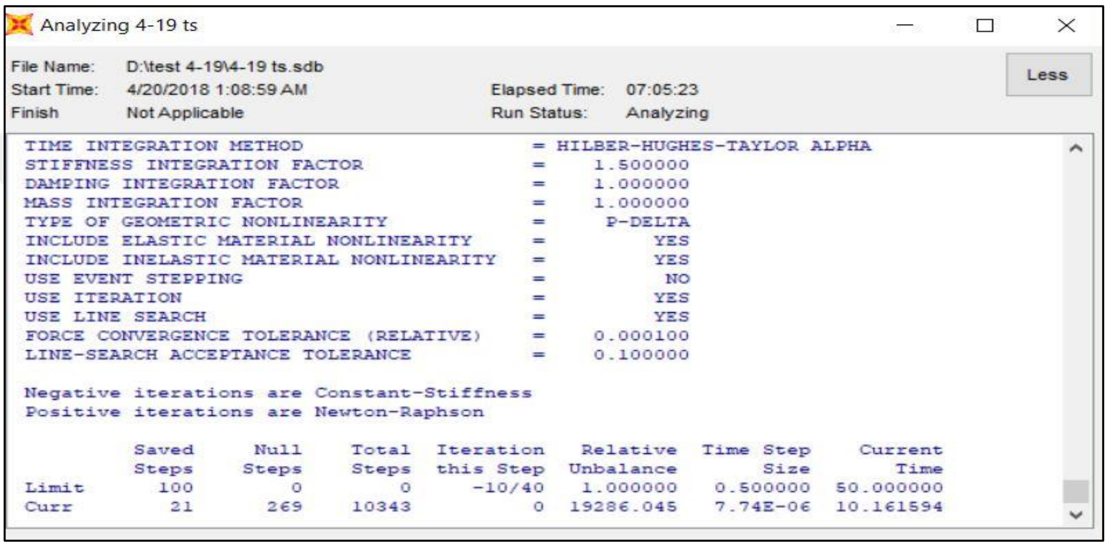


Figure 4.1: Time step size equals $7.74 * 10^{-6}$

As the value of **α** was decreased, it was noticed that **α** has provided what can be described as recovery mechanism to the solution so at events (like hinges development and the related force redistribution and energy dissipation) some iteration occurs and the time step get smaller and smaller to achieve convergence but after that the time step start to regain its original size while still achieving the convergence, and the analysis is faster by far.

At (**α=-1/3**) the analysis is fast compared to other values but as most of events points still could be noticed through iteration and smaller time steps, some energy seems to be lost due to damping of wide range of frequencies, which mean the analysis is less accurate.

It is recommended to start from (**α=-1/3**) just to determine when the critical events happen and then increase it to be close to 0 as much as possible and in the same time still providing the recovery mechanism after events occurring and monitor the behavior of the solution to make sure of it.

While this topic need more in-depth research and investigating because of the lack of clear understanding or definition in the literature, the five values were tried and (-1/48) was the most suitable to provide as close as possible to (0) (best accuracy possible) and in the same time a recover mechanism for iteration process (bigger values were tried but the solution couldn't recover the original time step size after events).

4.4.2 Recommendation to improve the performance of the structure:

These recommendations can be used for different types of structures with some modifications, even they are focusing on industrial building:

1-Provide a lateral load resisting system suitable for the building with clear load path.

2- Abide to seismic codes limitation for thickness and members sizes.

3-Provide a well-arranged diaphragm with enough rigidity in the floors and the roof to secure the load distribution to vertical elements of the lateral load resisting system.

In Figure 4.2 the primary roof braces arrangement which was not enough to meet performance requirement.

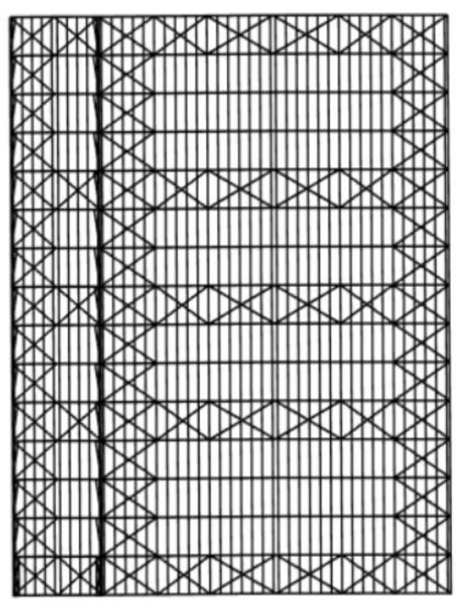


Figure 4.2: Primary roof braces arrangement.

In Figure 4.3 the modified arrangement which was adopted to assure good diaphragm behavior for the roof and provide lateral support for rafters

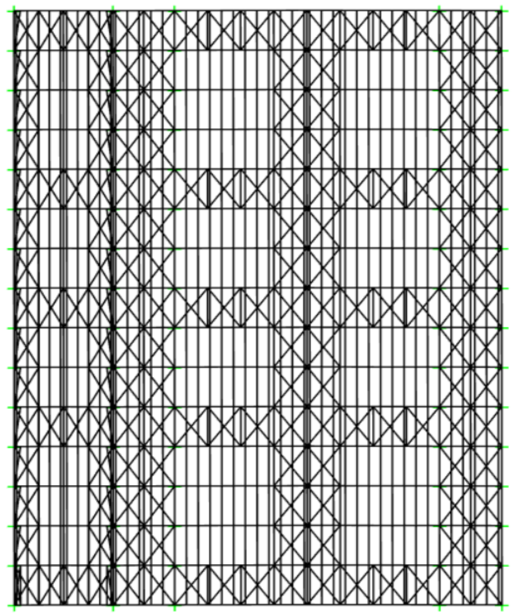


Figure 4.3: Modified roof braces arrangement.

4-Make sure that lateral resisting system is well connected so that the frames work together.

In Figure 4.4 unconnected frames, Figure 4.5 shows hinges development as a result of not connecting them under scaled record (DSP).

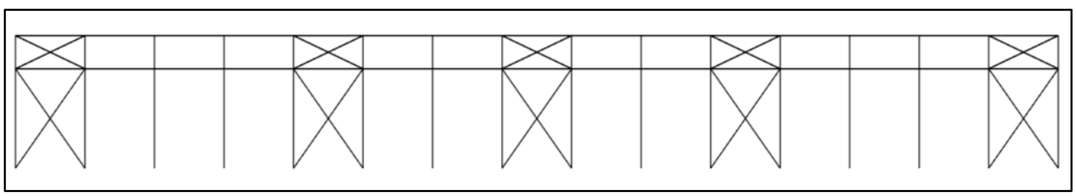


Figure 4.4: Primary brace system arrangement.

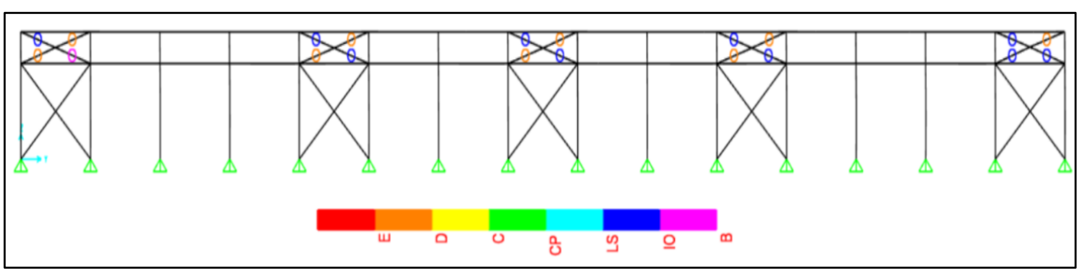


Figure 4.5: Performance of the unconnected brace frames under DSP.

After connecting and changing sections of the surrounding elements to account to plastic behavior (no hinges developed in braces after introducing this arrangement) as shown in Figure 4.6.

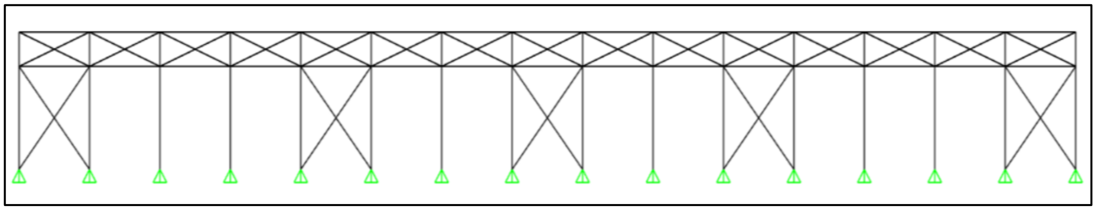


Figure 4.6: Modified brace system arrangement.

5-Check the structure under plastic design procedure to assure the elements are suitable and energy dissipating mechanism (plastic hinges) is occurring as supposed.

6- Try to avoid using tapered elements unless accounting to their resulting behavior (as creating weak points which would transform into hinges).

After several modifications and applying these recommendations, the structure met the performance criteria of Immediate occupancy (result under each record are shown in Appendix B), Figure 4.7 shows the result under CPE record as an example.

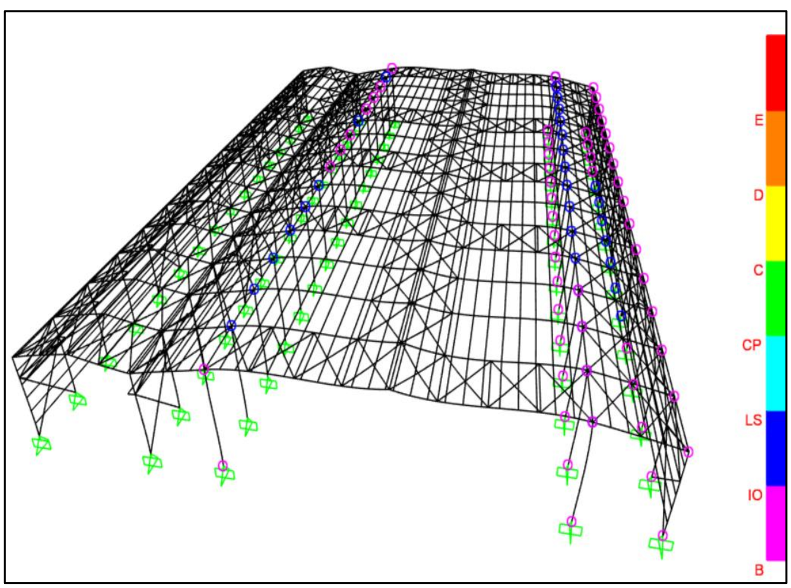


Figure 4.7: Structure performance under CPE record (achieving IO).

It important to mention that Performance-based design is not a series of steps but a philosophy, to determine an intended performance and to modify the structural system to achieve that level, considering the post-earthquake situation and loss in property or the needed repairs to put it back in function, the targeted performance is mainly determined by the owner but need deep understanding and expertise from engineer to implement it.

4.5 Base Plate

As mentioned in design part we found that common arrangement of straight line of anchors is not equally distributing of the forces so we suggested a curved arrangement which made them almost equal, we tried to develop a formulation that would be applied in general but we found that this issue need further investigation in the future.

4.6 Kocaeli Earthquake Spectrum

As could be noticed in Figure 2.14 containing Kocaeli earthquake spectrum, the shape of the spectrum is different from commonly used spectrum which means that building designed using currently code defined spectrum need to be assessed to assure they can sustain the forces resulting from event with characteristic spectrum like Kocaeli spectrum, this specially include medium to high rise building with periods more than 0.7 sec.

A spectrum needs to be developed from local monitoring stations accelerations records spectrums to best reflect the design requirements for structures in turkey or some modification for the currently used method to account for situations like Kocaeli earthquake.

4.7 For Future Studies

1- Further investigation for Kocaeli earthquake spectrum and other local records to address their nature and account for it in Turkish seismic code.

2- Further investigation of base plate's anchors arrangement, including experimental and simulation to determine the optimum arrangement.

3-Further investigation of HHT method (α) values to determine the optimum value according to structure properties, so it can be introduced instead trial and error method currently adopted.

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APPENDICES

Appendix A: Scaled accelerations

Appendix B: Performance result due to each pair

Appendix C: Earthquake reports (CD:Addional material 1)

Appendix D: Elements design and other calculations (CD:Additional material 2)

Appendix A

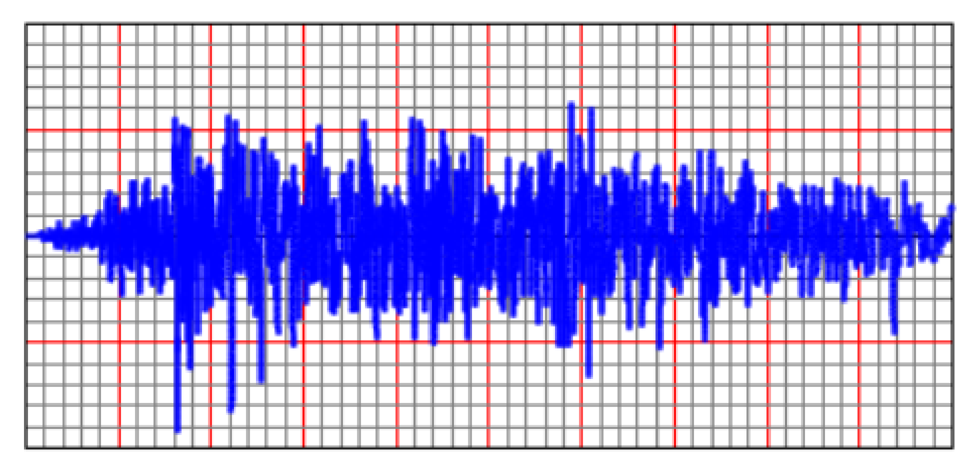

Figure A.1: NPS000.

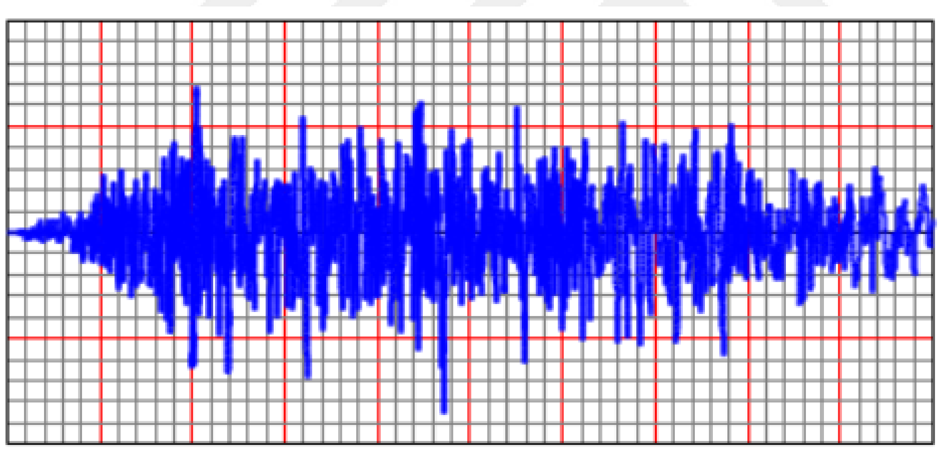

Figure A.2: NPS090.

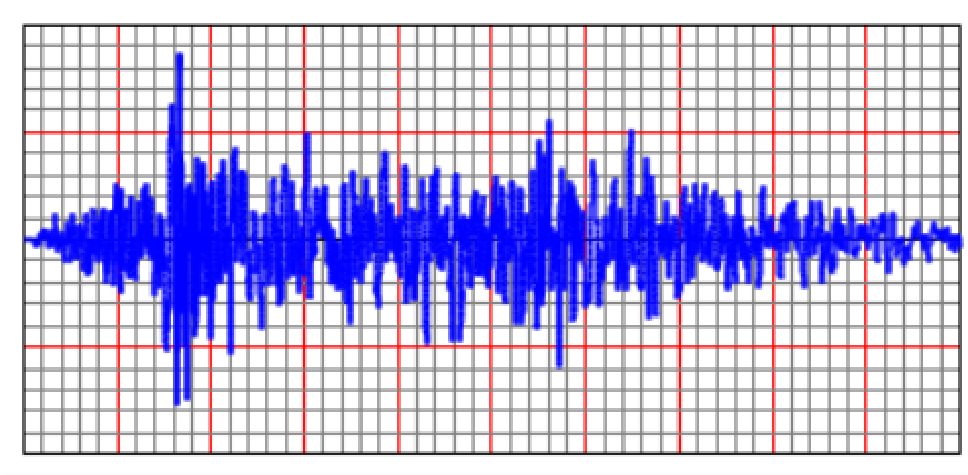

Figure A.3: MHV000.

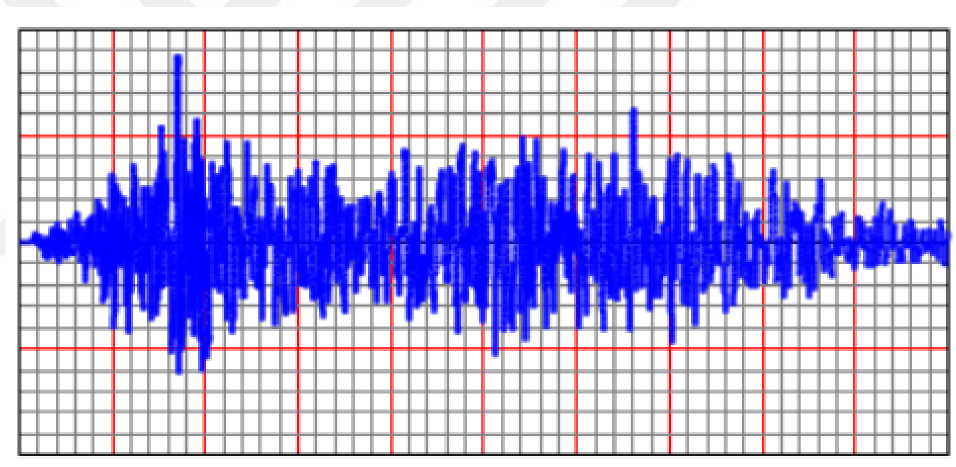

Figure A.4: MHV090.

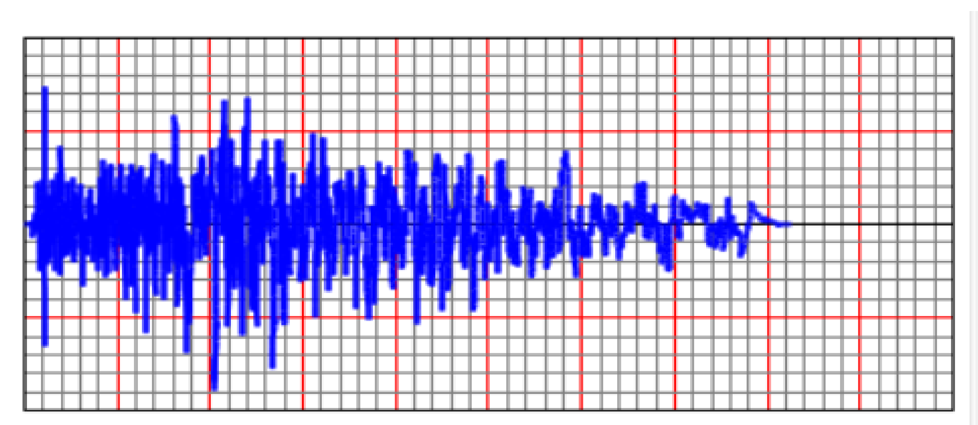

Figure A.5: LAMONT362E.

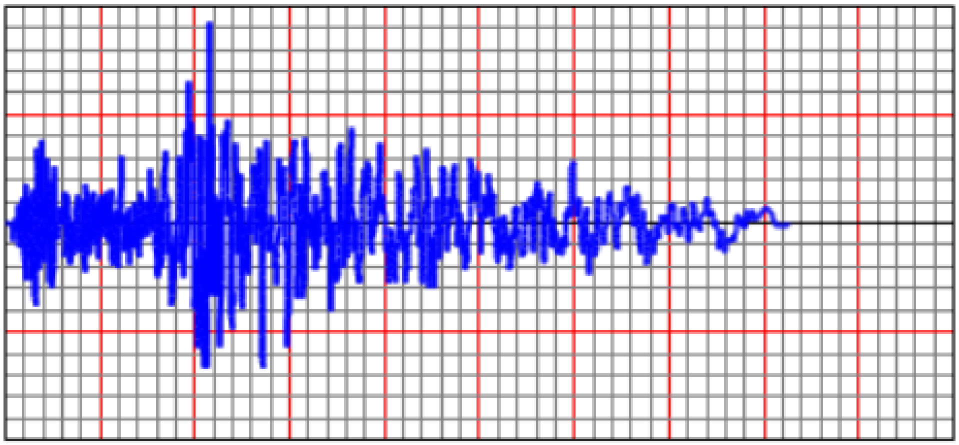

Figure A.6: LAMONT362N.

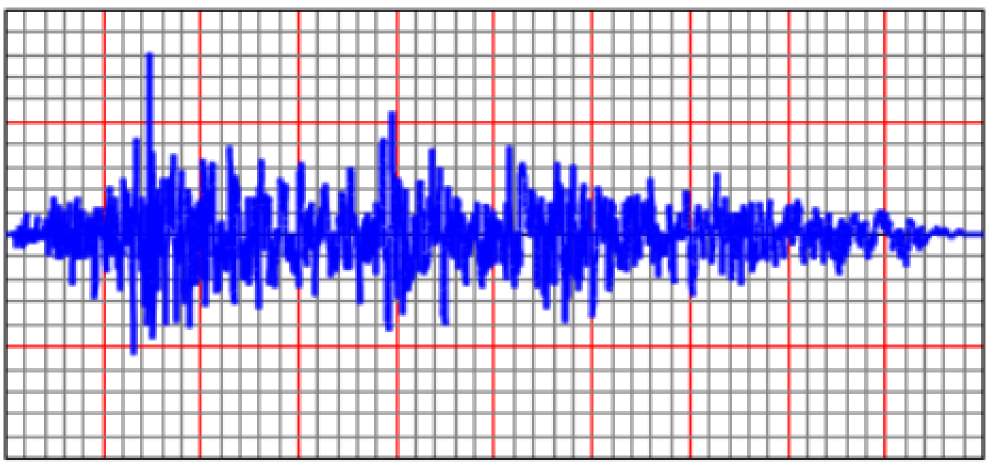

Figure A.7: DSP000.

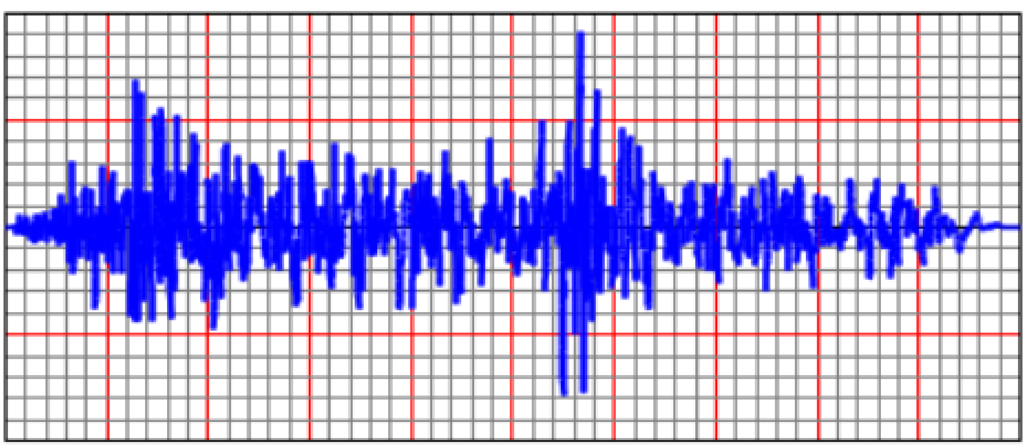

Figure A.8: DPS090.

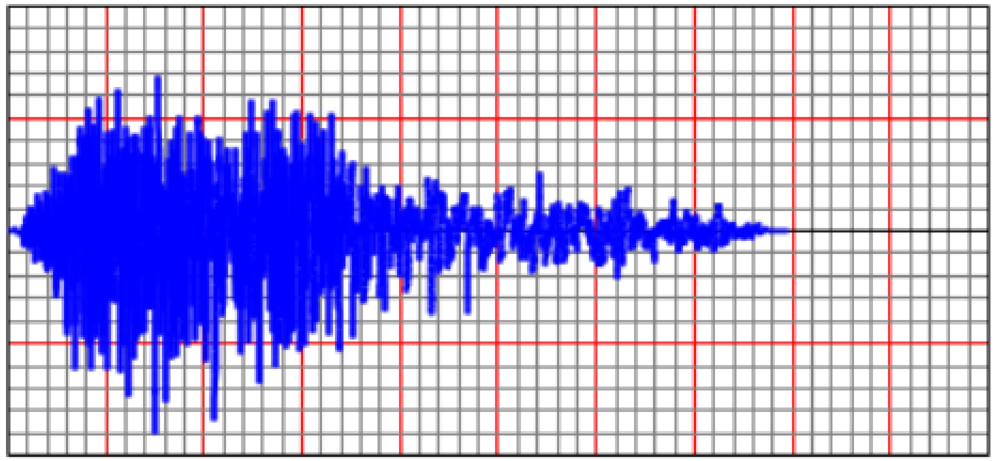

Figure A.9: CPE147.

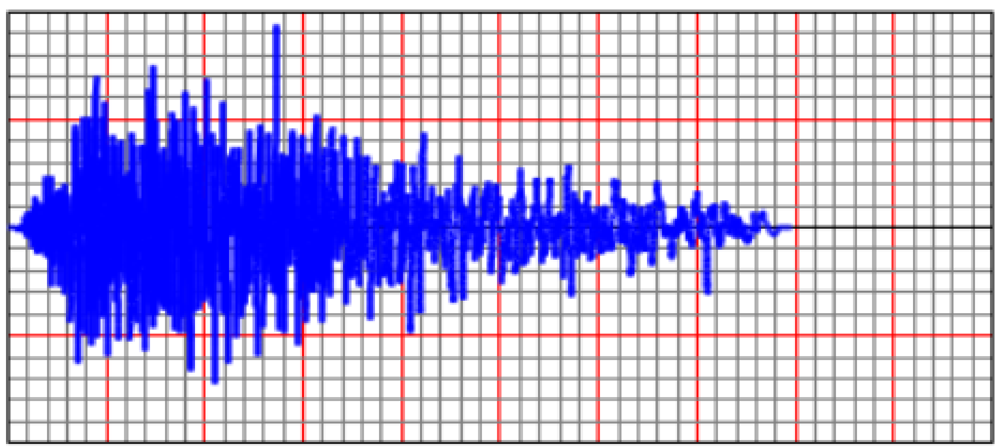

Figure A.10: CPE237.

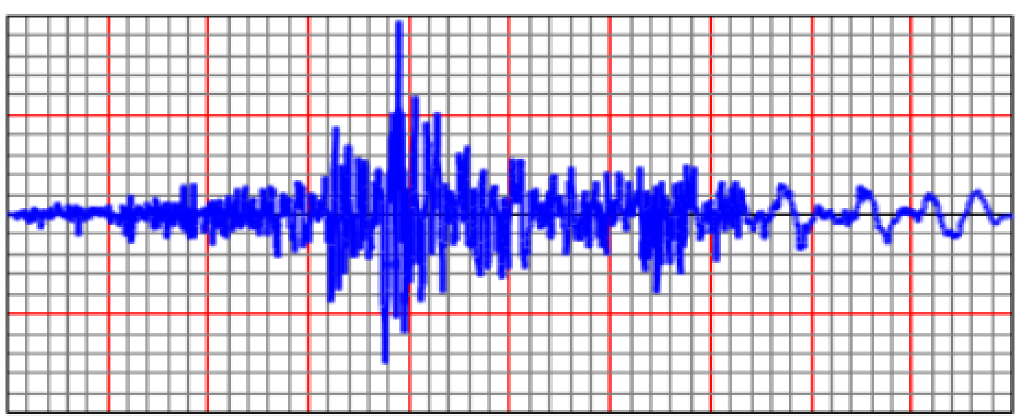

Figure A.11: BRS000.

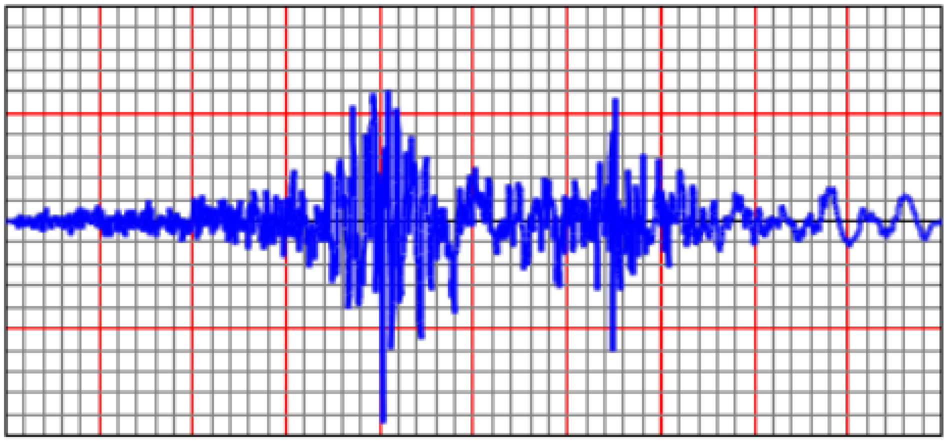

Figure A.12: BRS090.

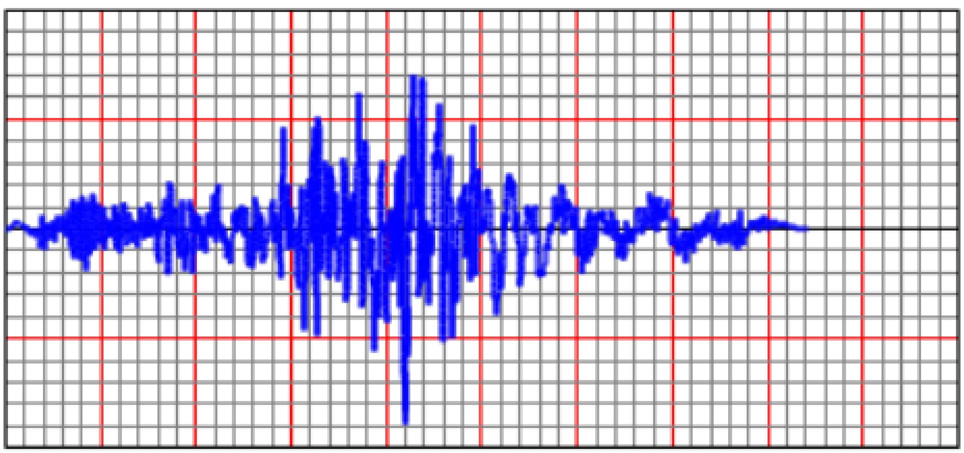

Figure A.13: ARC000.

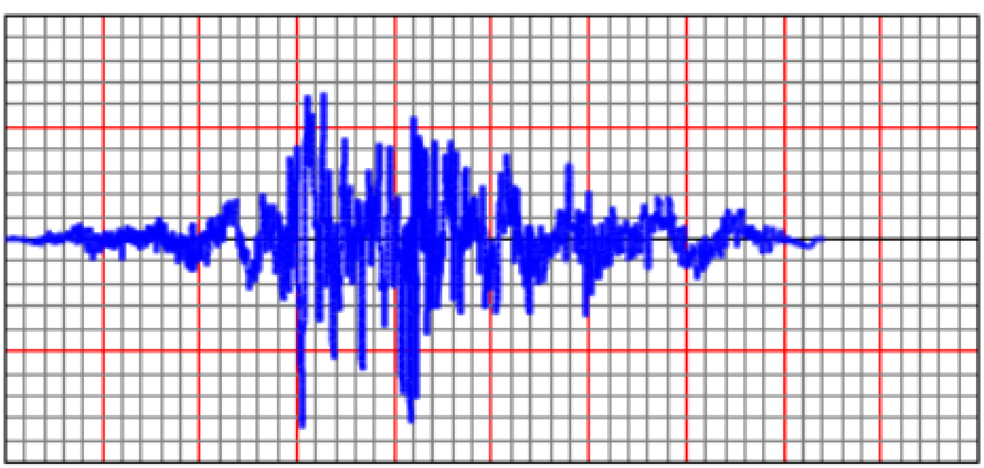

Figure A.14: ARC090.

Appendix B

Performance result due to each pair

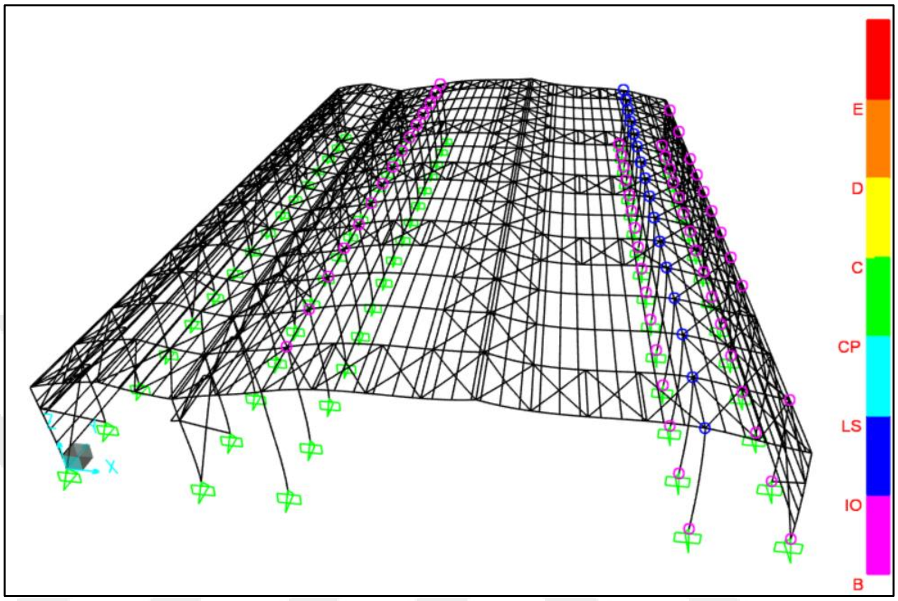

Figure B.1: DSP.

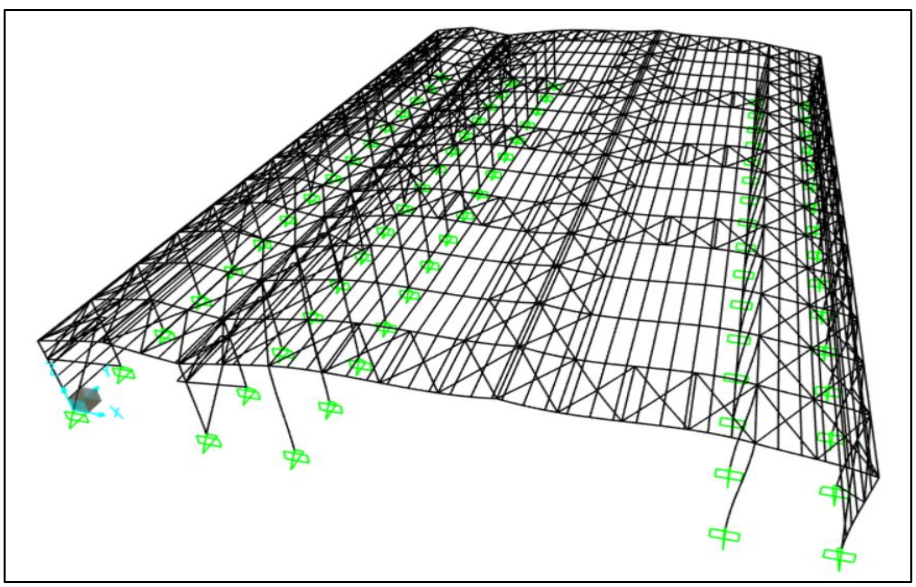

Figure B.2: ARC.

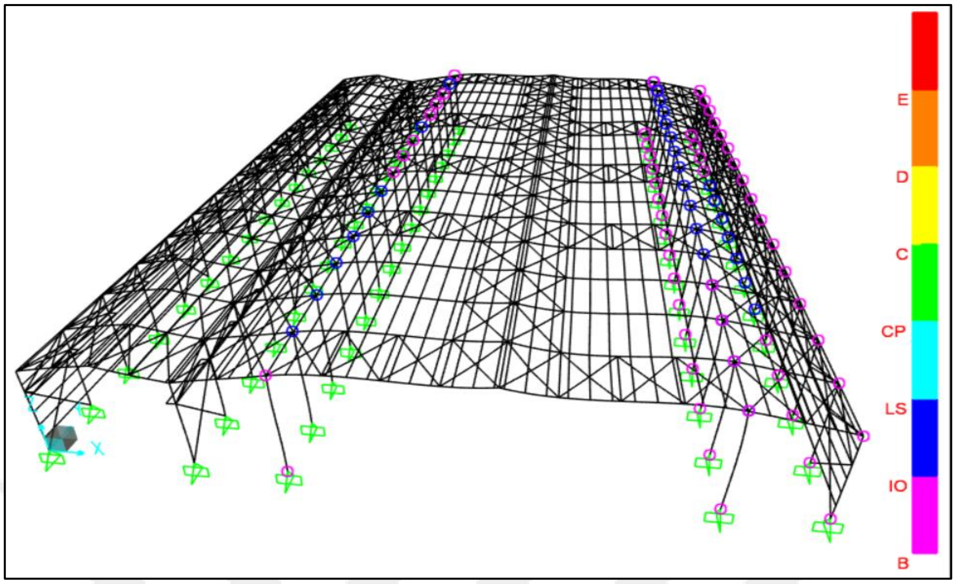

Figure B.3: CPE.

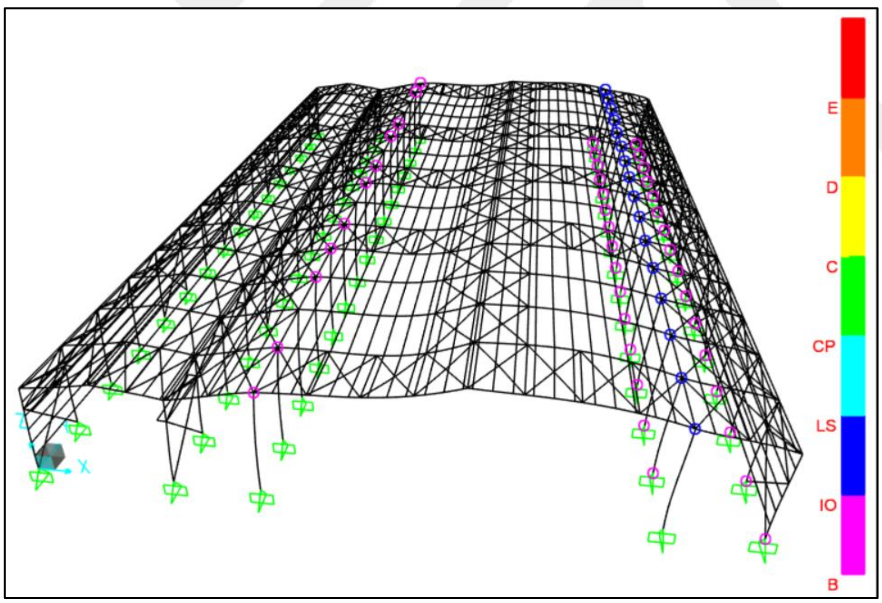

Figure B.4: LAMONT.

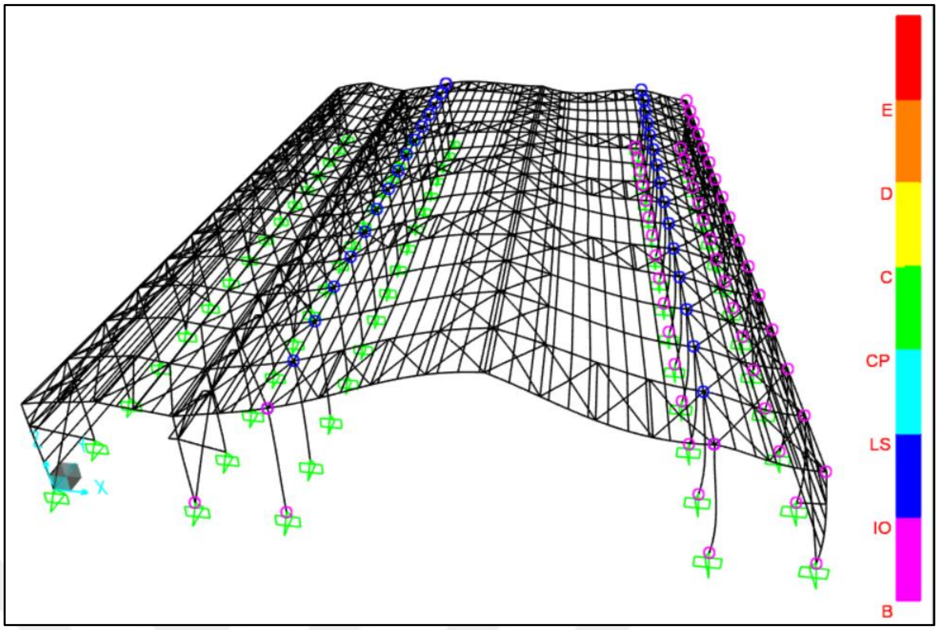

Figure B.5: MHV.

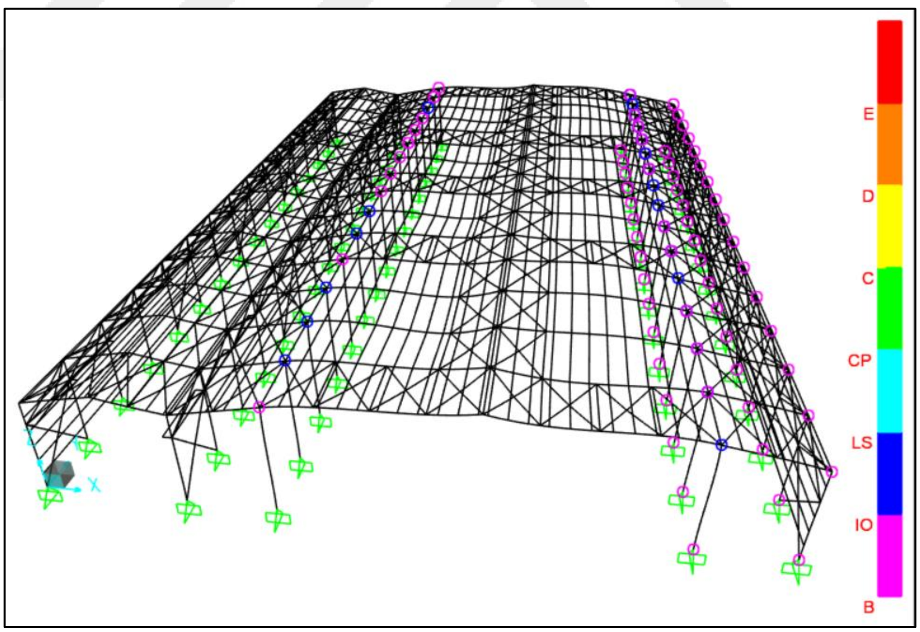

Figure B.6: NPS.

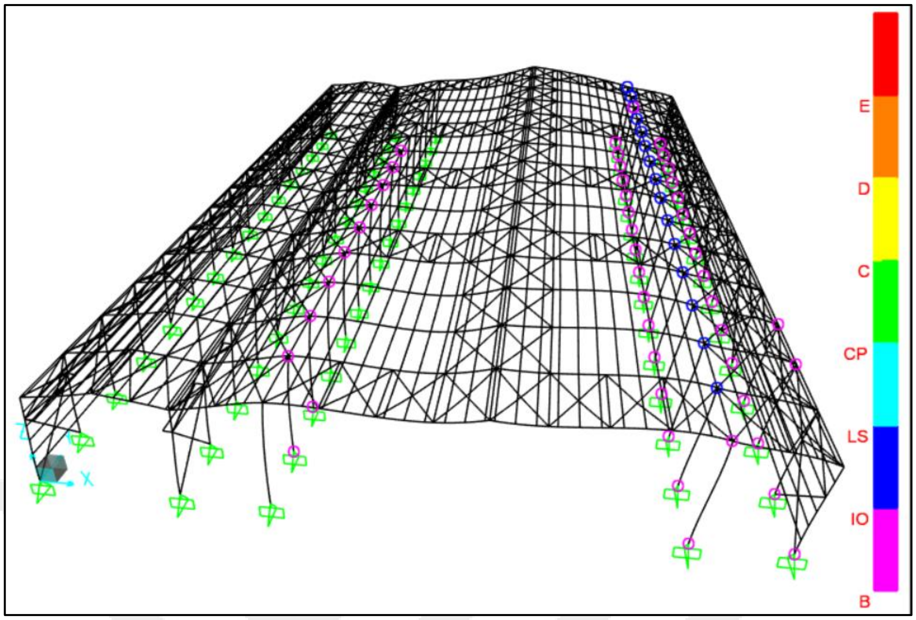

Figure B.7: BRS.

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