Critical Review of Steel Column Base Plate Design Codes and Their Relevance to United Arab Emirates

مراجعة نقدية لرموز تصميم صفيحة العمود الأساسي وصلتها بالإمارات العربية المتح

by

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ABSTRACT

Structural engineers are often posed with a question for an economical design of a structural building which would decide the overall economic and the time concern in a construction. If the design is at optimum by choosing a suitable code the involvement of the materials would be less which will help to decrease the pollution caused in the environment as well. In short where structural steel works are being used with other structural materials the importance of various connection plays to be indispensable. One of which the steel column base plate connection that would perform adequately for the specified demand. This is the most critical element in the entire structure which would carry the load of the entire building through the columns to transfer to the base plate and then to the ground and ensuring the stability of the structure. Hence this dissertation intends to bring about a requirement of workability such as the easy and efficient construction, working the connection with high loads and deformation with sufficient capacity ensuring least cost, maintenance and with long durability with an economical design code. In the United Arab Emirates where the country is home to all the high rise structures will need this aspect as the cost involved is at high steak. The three codes which are thus reviewed in this are the Eurocode, American and the British standard codes emphasizing more on latter two codes (British and American). The involvement of the computer analysis supporting the work will provide much understanding to choose which code is more suitable for a desired project for the municipalities and consultancies in U.A.E.

نبذة مختصرة

وغالبا ما يطرح مهندسو الهيكلي مع سؤال لتصميم اقتصادي للمبنى الهيكلي الذي من شأنه أن يقرر القلق الاقتصادي والوقت الكلي في البناء. إذا كان التصميم على النحو الأمثل عن طريق اختيار رمز مناسب ، فإن مشاركة المواد ستكون أقل مما يساعد على تقليل التلوث الناجم عن البيئة أيضًا. باختصار حيث يتم استخدام أعمال الفولاذ الهيكلي مع المواد الإنشائية الأخرى ، لا غنى عن أهمية لعبات التوصيل المختلفة. واحد منها اتصال لوحة قاعدة العمود الصلب التي من شأنها أن تؤدي بشكل كاف للطلب المحدد. هذا هو العنصر الأكثر أهمية في الهيكل بأكمله والذي سينقل حمولة المبنى بأكمله من خلال الأعمدة للانتقال إلى اللوحة الأساسية ثم إلى الأرض وضمان استقرار الهيكل. وبالتالي فإن هذه الرسالة تهدف إلى تحقيق متطلبات قابلية التشغيل مثل البناء السهل والفعال ، والعمل على الاتصال بأحمال عالية وتشوه بسعة كافية تضمن أقل تكلفة ، وصيانة ، ومتانة طويلة مع رمز تصميم اقتصادي. في دولة الإمارات العربية المتحدة ، حيث تعد البلاد موطنًا لجميع الهياكل الشاهقة ، ستحتاج إلى هذا الجانب نظرًا لأن التكلفة المترتبة عليها مرتفعة للغاية. الرموز الثلاثة التي يتم مراجعتها في هذا هي الكود الأوربي ، الأمريكي والرموز القياسية البريطانية التي تركز أكثر على الكودان الأخيران (البريطاني والأمريكي). ستوفر مشاركة تحليل الكمبيوتر الداعم للعمل فهماً كبيراً لاختيار الكود الأكثر ملاءمة للمشروع المطلوب في البلديات والاستشارات في المملكة المتحدة.

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SYMBOLS

EUROCODE

$N_{sec,Ed}$	is the applied forces in stub flange
t_p	is the base plate thickness
s	weld leg length to the stub flanges
f_{ys}	the shear stub yield strength
A_{vs}	shear stub area
f_{jd}	the joint's design bearing strength
d_f	concrete foundation depth
α_{cc}	0.85 as per BS EN 1992-1-1
Υ_c	concrete material factor essentialy it is equal to 1.5 as per UK
c	is the base plate width limit
l_{eff}	an equivalent of T-stub's effective length
t_{pl}	base plate thickness
f_y	base plate yield strength
m_x	That distance b/w bolt centerline and the fillet weld to the flange of the column
n	bolt numbers
$F_{t,Rd}$	a single bolts tensile resistance design

AISC

e	eccentricities
M	the max. moment produced
P	applied load

μ	friction coefficient b/w steel and the grout/cement
A_{brg}	The region where the contact occur column and base plate over concrete.
P_a	The anchorage factored axial load (external) where it is –ve when there is compression and +ve when there is tension. When the case where P_a is –ve it is recommended to verify the existence of P_a in the occurring shear force.
c_1	edge distance accounted in load direction
f_c'	compressive strength of concrete
l	the depth of embedment
d_o	the diameter of the anchor rod
ψ_5	1 when all anchor are to have the same load
ψ_7	1.4 when the concrete is intended to be with proper supplementary reinforcements or uncracked
N_{cp}	single anchor's nominal concrete breakout strength
h_{ef}	effective embedment length of anchor
f_p	The bearing stress in b/w concrete and plate
B	the base plate width
F_y	The given yield stress of the material of the plate
t_p	the thickness of the plate
Φ_b	strength reduction factor (bending)
Ω	safety factor (bending)
d	column flange width
t_f	thickness of column flange
D	major diameter

n	Number of threads/in.
N	length of the base plate (in)
B	width of the base plate (in)
b_f	width of the column flange (in)
d	column overall depth (in)
n'	Theory of the yield line distance of cantilever from the column flange or web (in)
P_a	the axial compressive load which is required by ASD
P_u	the axial compressive load which is required by LRFD
φ	flexure resistance factor
Ω	Factor of Safety
F_y	base plate minimum yield stress as specified (ksi)

BS 5950

p_t	the bolt tension strength
p_{yp}	the baseplate design strength
w	the pressure produced under the steel base plate as per the assumption of the uniform distribution of the pressure all over the effective portion.
c	is that largest perpendicular distance between the face of column and the effective portion of the steel base plate.
t_p	Maximum or usually the flange thickness of the section as in the column.

CHAPTER 1: INTRODUCTION

1.1 Research Background:

The design of a building is a concern for its stability and safety. Structural engineers strive to achieve such an agenda through investigation and analysis of the structure. This approach with the given conditions at its crucial scenario would explain the behavior of the building during its design life course, which would offer the structural engineers a sense of the structural behavior thus allowing to provide the required detail and design for the structural building. However, the engineers would need a design aid to accomplish standards that would conform to the respective region where the construction has proposed, which would commemorate uniformity and consistency of design philosophy in the country. Some nations may not have the building design codes of their own. However, they tend to use the other approved international building design codes to serve the purpose and would want the structural engineers to strictly abide by the rules and the regulations of the design codes. Hence the great nation of the United Arab Emirates encourages to use two international codes which are the American and the British standard codes for the design.

Steel is the most trusted material around the world, and it’s no surprise why many buildings built with it. The reason is that steel structures are sustainable, durable, ductility, versatility, and can be erected quickly at a minimal amount of money. That is the reason for steel to be reigned supreme in the construction industry for over a century. Thus a critical interface of any steel structure and concrete foundation is the column base plate which ensures the safety and stability of a structure. These types of connections made in building structures to support the loads due to gravity and integrate itself as a part of the lateral load resistant system wherein it is affected by dynamic and fatigue loads. Hence critical elements of a building need to be designed with the utmost care, and economically are depending on the approach of the design method used.

1.2 Research Significance:

Building design codes are established to maintain the structure constructed to be in harmony with the conditions in that region to ensure safety and prevent the loss of lives. Often structural engineers are faced with an issue of choosing the best-suited code for the region as there are many and switching between codes are unpractical and unethical during the designing stage. In the nation of the United Arab Emirates, the two followed codes and approved are the American and British standard codes which have been discussed earlier in the section. There should be a system that needs to be maintained to ensure the uniformity of the design methods followed to enable a fundamental understanding of building workability. Since the importance of a steel structure lies in its base plate connection, it is essential how well the design of it is carried out, transferring the load to the ground efficiently. So the designing has to be done with extreme care and surety. To achieve such a task, a structural engineer has to follow relevant design codes to safeguard the stability and life of a building. However, the design is severalty effected by the codes chosen hence making it very important to understand the similarities and differences among the codes to arrive at the most economical design although they tend to comply with a similar philosophy of design.

1.3 Research Objectives:

This research work anticipates to critically assess the behavior of a steel column baseplate of a structural building when different renowned building codes used for its design through a comparative study with the earlier literature findings and the parameters used by the two codes. The comparison hence made would provide a significant understanding of an economic and safe design to be chosen for the steel column base plate in an average building scenario to improve the uniformity in design that would be suited for the conditions in the U.A.E. However, the research also includes a general comparison of some of the other international code other than the American and the British such as the Eurocode. It has brought to the attention of the reader that this work intends to purely focus on the contrast of the American and the British design codes which would bring about to which code is opposite and propitious to conditions in the U.A.E.

1.4 Research Methodology:

The intended method of approach for the research is preferably carried out in a distinct technique.

Primarily a general narrative is made for the three codes under the study viz. American (AISC, ASCE), British (BS 5950, BS 6399) and Eurocode (EC-1, EC-3) where the significant critical limits include its shear, moment, and the flexure. A detailed parametric comparative study about the used empirical formulae has also been carried out to comprehend the results of design output during the change in the parameters for example how the design of the connection would vary load transmission from the entire structure to the foot without affecting the stability and the behavior of it through shear, flexure and moment. The study then constricts to the two relevant international codes mostly used in the country of U.A.E., which are the American and the British design codes. The approach of design and the related parameters used for the equations of the respective codes enables to understand of a designs limit state through the differences that occurred in their output. Which is then followed by accurate modeling of a simple structure and analyzing it with the two codes for a further understanding. STAAD, RAM Elements & Master Series, which is a commercial software widely used in the U.A.E have used in the analysis. An essential case study on steel column baseplate is made to review workability during the construction period to make a better design so forth. The methods herein are expected to narrow down to that code which would be efficient and economical for constructional ethics and practicability in the nation of United Arab Emirates.

1.5 Research Challenges:

The course of development in technology in past decades been tremendous so as the research behind them. The design codes thus formed for the building construction is abundant. Almost every nation has its building code to be followed, as discussed in the previous sections. Hence the time was the primary constraint to read and review the relevant codes which would comply with the U.A.E. norms. Variety of the design code provisions for the steel column baseplate also had to meet with a limited amount of literature available which would necessitate the agenda of the comparative study. The critical and most significant to relate the relevance of this research to conditions of the United Arab Emirates and to arrive at a quick solution explanation in the forthcoming chapters.

1.6 Organization of Dissertation:

The research work intends to bring about the most economical design code used for a structural building after comparing the similarities and differences of some of the significant codes used internationally. The critical aspects and their respective parameters would be compared and bring about that design code which will have the upper hand. However, in U.A.E., the American and the British codes have more relevance the computer analysis done shall be emphasized mainly.

An appraisal of the limited available literature of the international codes for design has been carried out in Chapter 2 while focusing on their major design provisions.

Whereas Chapter 3 deals with a general comparison of the principal international design codes which are applicable in the world concerning their significant design philosophies and the effects of the forces such as the shear, moment and axial over the assembly.

Chapter 4 is the section where the review on parameters and used empirical formulae of each code made to understand how it affects the geometry and the design of the steel column' baseplate.

Chapter 5 is an' overview of the comparison made through the theoretical approach discussed in the previous chapter relating it through a practical method using computer analysis to make the reader understand further with discussion and recommendation.

Chapter 6 briefs about the most used type of anchor bolts and why it has chosen in the nation of U.A.E. has shown through a case study. A short description of the advancements has also been appraised in the chapter as well.

The final chapter 7 depicts the conclusion of the research work of the effects of using different design codes on the steel column base plate.

CHAPTER 2: GENERIC DEPICTION OF LITERATURE ON COLUMN BASE PLATE WITH STEEL BUILDING DESIGN CODES

2.1 Steel Column Base Plate

2.1.1 Introduction:

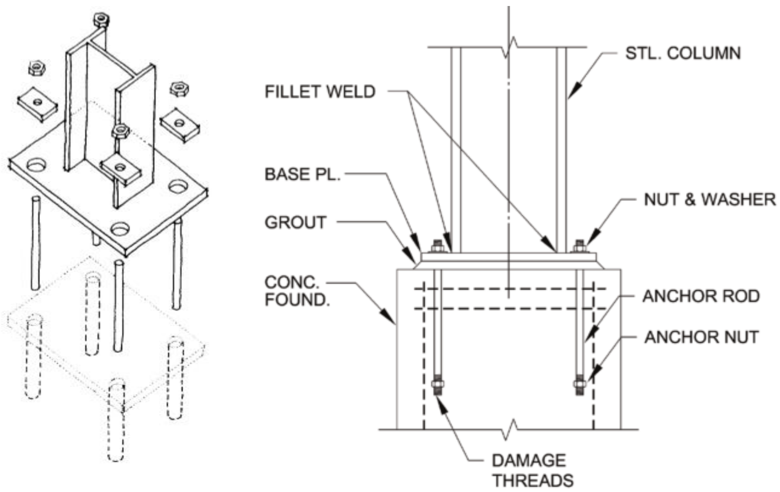
Baseplate connections are omnipresent element in any steel structure in existence and are in diverse types i.e, at the foot of columns of a building, non-building structures like silo, tanks and equipment at bottom of the steel plate wall, as well as the anchoring of a non-structural component to a structural elements like slabs and floors made of concrete. Irrespective of the use the base plate connection have some shared feature which is the base plate welded to a primary structural member, concrete foundation, anchors and finally the grout. There two types of steel column baseplate connection used, which are the fixed and the pinned type as per requirement. In general, the part of work would be focusing on the steel base plate connection in structural buildings.

2.1.2 Assembly:

The most fundamental part of the assembly is attaching of steel base plates to the required concrete or masonry components. It's that part of the concrete-steel connection which does the connecting. The steel column base plate provides the transfer of different types of loads such as tension and shear forces from the steel structures. An optional piece is the shear stud or the key, 'chairs' to increase the stretch length of the anchor, shim plates or leveling nuts as the leveling components and baseplate stiffeners. The components of the steel column base plate are as follows:-

- Steel column
- Concrete base (pile cap, raft, pad or strip)

- Base plate
- Stiffeners if required as per design
- Threaded rod, grouted in a drilled hole
- Oversized holes in B.P / Pockets
- Plates or angles
- Plate washer
- Dry pack or Grout



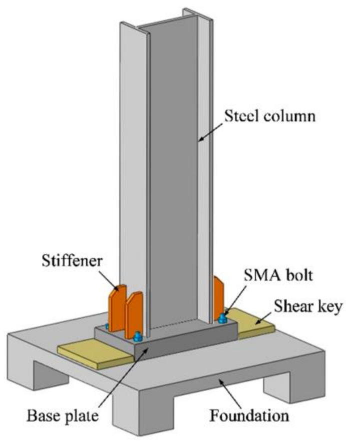


FIGURE 2.1: Representation of A Basic Steel Column Base Plate (SCI Publications, AISC Manual, and Engineering Structure Journal)

The construction in an era of modern engineering has a rapid pace, which calls for different requirements. So the system has evolved accordingly to serve the needs and purpose of a structure. In addition, hooked rods might be cheaper than the threaded rods or headed rods with a nut, but its recommended, when there is a calculated tension force, hooked rods not to be used as they have a limited pullout strength and there is a tendency to slip under tension especially when oil remains on the rods due to the thread cutting.

There are two types of anchoring which have introduced for the construction community such as:-

1. Cast-in-place: These types of anchors are the simplest and most durable which have standard hexagon heads (embedded end) or threaded sleeves or welded flange and are fit before the concrete placed. The load transfer principle of the system is mechanical interlock, which is the cast-in-place anchors tend to transfer the shear or axial or tensile loads with the embedded head by bearing pressure to the concrete. Other than using the cast-in-place anchors for building anchorage, it also serves to constrain the machines to a concrete floor.

Depending upon the requirement and capacity needed, the following are the different types of Cast-in-place anchors used:-

- Lifting inserts: These can be threaded rod which used for lifting or pre-stressed RC beams.
- Anchor channels: Used in precast concrete connections. The channel T-shape screwed to it to transfer the load to the base material.
- Headed Stud: These are the type of anchors which have welded headed stud with steel plates
- Threaded sleeves: These have internal threads in a tube anchored back to the concrete.

2. Retrofit/Post installed: A typical retrofit/post-installed anchor installed after when the concrete has hardened in any position. This type of anchor can have adhesive, grouted, undercut, or expansion.

- Undercut anchor: This type of retrofit anchors transfer of tensile loads to the concrete is by bearing of an expansion device against a bell-shaped enlargement of the hole at the base of the anchor. The components of the undercut anchor consist of an expansion device, a sleeve, and a threaded rod.
- Adhesive anchor: A type of retrofit anchor that transfers the tensile loads to the concrete through bonding along the length of the embedment of the anchor. It’s a threaded rod

installed in a hole where its diameter is about 1/6th to 1/8th inch larger than the diameter of the rod.

- Grouted anchor: Anchor which has a headed anchor with a diameter about 1/1/2 inches larger than the diameter of the anchor which installed in a hole filled with a non-shrink grout which usually contains Portland cement, sand, hydraulic cement, and various chemicals to promote the reduction of shrinkage. These types of anchors transfer the tensile load to the concrete by bearing on the anchor head, and by bind along with the grout/concrete interface.
- Expansion anchor: Tension load transferred to the concrete employing friction amongst anchor as well as concrete at the base of a hole drilled in it. In Compressive reactions generated during the opposition to the movement of expansion, mechanism results in the friction force at the embedded end of the anchor. Those anchors which are Wedge did not suggest as they have to be tensioned to locked into the device. At the time of erection, which causes the column to move can cause the wedge anchor to loosen.

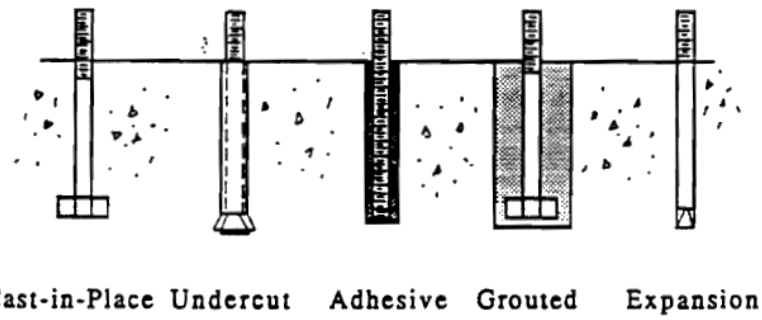


FIGURE 2.2: Types of Anchor Bolts (SCI Publications)

Hooked type anchors used extensively, but it lacked the pull-out strength compared with those rods which are threaded or rods with headed along with nut for anchorage. Hence the endorsed practice nowadays is the use of threaded or headed rods along with nut for anchorage.

The selection of materials and detailing with the design of base plate and anchor rods is significant which would affect the cost of erection and fabrication of a steel structure, moreover the performance of the building under the load as well.

As discussed in the earlier chapter, the steel column base plate is the essential part of a steel structure as it governs the initial stiffness, but the grout employed for the convenience of the erection of the column. In most column base connection concrete grout is used to enable the construction and that the full contact amidst' the baseplate and the concrete pedestal ensured. It also improves the behavior of the connection of the column baseplate. However, there is a scarcity of understanding of the shear strength contribution of connection of the column base. Were the connection is without grout the load applied would counter by the shear and bending forces in the sole anchors. The plastic hinges developed in anchors achieved the capacity of the connection, which then followed the connection mechanism failure.

There would be an increase in ultimate displacement, and the shear capacity only if the thickness of the grout increased. There is an increase in the severance if the grout utilized for the connection as it develops grout struts and there is a rise inconsequently the obligatory quantity on a plastic hinge in the anchors for the failure of mechanism which in turn develops high tension in the anchor rods of connection with the grout. The base plate rotates from its front side with surface friction wherein the grout pad is stanched even though there is no applied axial force this is lead due to the unequal distribution of forces in the anchors beneath the shear load applied. The increasing tension force that results in the vertical component, which enhances the friction force by the action of clamping in which the design described above codes have overlooked these positive effects.

In an anchor rod where the force developed is purely depended on the bearing and the exposed length between the grout and the anchor where they have an essential part in the ultimate strength of the desired connection. The shear strength and the lateral deflection is reduced and increased respectively, as there is a subsequent increase in the length exposed. The strength of the grout depended on the following factor: Relative humidity, Temperature, Water/cement ratio, Fine/coarse aggregate ratio, Raw materials. If any of these factors are inaccurate, then it affects the grout quality or its strength, which can be lower than expected. Cautioned that the area of bearing b/w the grout and base plate is effected majorly by the wrong method of placement, lousy mixing of the grout, use of weak grout or it can even be the leakage of the grout.

The purpose of grout placement under the base plate is to provide lateral support for the anchors beneath the shear force. When there is increase in the shear load, the anchors experience a lack of confinement because of the grout crushing. Due to the thickness of grouting, it effects peak displacement in about lateral direction and the strain hardening significantly beyond the range of the elastic of assembly. When thin or thick grouting used, there is an increase in the shear capacity, respectively.

The grout thickness to be provided under a connection is purely on the design and the practical knowledge. Hence for the flowable hydraulic cement grout, the thickness which followed at the minimum is 25mm where it also should be noted that placed realistically. For each 300mm flow length, the thickness is required to be increased by 13mm up to max. of 100mm only if the flow length is more substantial than 300mm.

There are two main reasons why the grouting enhances the capacity of shear in a base plate assembly. Firstly in the connection’s elastic range, there is a strut (concrete) formed in the grouting layer so this phenomenon laterally restricted/restrained the anchors. Secondly, there is this development of friction amid the grouting pad and the column baseplate. Strength of the grout has minimal effect overcapacity of shear for base connection predominantly when the grout used is thin; hence, we can calculate the shear capacity independently.

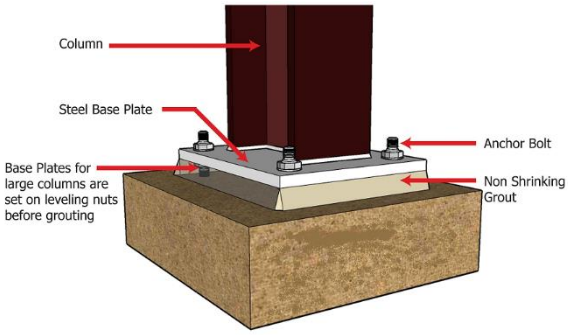


FIGURE 2.3: Representation of Grout (SCI Publications)

According to the user application, the grout is of different types. It is the volume of the grout used to ensure the permanent and complete filling the in b/w space of a footing and the base plate which is a first feature that affects the transfer of the loads to the concrete from the column base. A simple plain grout has fine aggregates, water, and cement as its contents, which can improve sufficient strength. If there is a chance of the grout to bleed and the possibility of shrinkage in the scenario, then there won't be full contact with the column base plate, therefore, to ensure the full contact certain additives used. The values recommended by the standards strictly not followed in the construction industry rather conservative values used for the non-shrink cementitious grout strength, for example, the typical material to be used for the grout is in the range b/w 48MPa and 56MPa as per the grout suppliers worldwide.

It is required by the AISC code to have a minimum of strength to be twice of the concrete pedestal to transmit the load safely to the concrete foundation from the superstructure whereas the ACI suggests having a compressive strength to be in the range of 35MPa and 55MPa typically which do not regard the concrete pedestal strength. Now the EC3 states it has the characteristics strength of grouting to have nothing less to 0.2*times of a concrete pedestal's strengths characteristics. The shear capacity calculated in both EC3 and ACI code is free of the exposed length of the anchors, which results in perversely to have the same shear strength even when the thickness of grout altered. However, the ACI tends to be less conservative predominantly for those substantial exposed length connections.

Typically used grouts for construction are the epoxy and the hydraulic cement grouts as per ACI standards. The hydraulic cement grouts mixture is identical to that of a plain grout like the water and the fine aggregate used in it and with additives like are used further to prevent bleeding or the shrinkage and known as cementitious non-shrink grout. These types of grout are preferred since it has the competency to transmit dynamic and static as well as impact loads.

As per the code generated shear strength design major damages to the concrete pedestal expected, which would alter the connection behavior. To prevent such damages, large edge distances b/w the anchors, and the concrete edge or even by reinforcing the pedestal is recurrently used in the engineering practice to elude the failure of concrete, for example, a shear breakout in the concrete. As the standards are concerned mainly about the failure of steel, in other a connection's capacity is not controlled by the failure concrete.

Beneath the applied shear load there is a significant increase in the lateral displacement which is high, and it may violate the effect of the induced forces in the steel column or the serviceability limit state because of the second order effect. However, the design codes consider the checks for the final state only by ignoring the sizeable lateral displacement of forces established in the connection assembly.

Note that the grout is one of the critical parts in the column base plate assembly however there is a shortage of research which would define the shear capacity affected by grout for the column base connection even though the layer of grout broadly used in almost all the base plate connection.

2.2 General narrative of the Literature on the Steel column base plates with Steel Building Design Codes

2.2.1 General:

Code is a set of rules and regulation which is documented as per the desired regions conditions building as well as it's a standard which is the major contrivance to be used for any design. It is the connecting bridge between a construction and a good design of a structural building. It guides a structural engineer to contemplate the requirements for the structure. Since there are many renowned international codes, the designer is restricted to choose that code which is approved and used by that particular region. An engineer would need to follow norms, and legislation intended and have to follow to retain the consistency of the design approach in the country. The building design codes are intended to provide a maximum life span of a building governing its maintenance and repair over the course. However, the intended load suggested by the codes over a building would be different in each design codes, thus varying the result in the column reactions. These reactions would decide the entire geometry of the steel base plate connection to the concrete and how it can maintain the provision of stability to the structure.

2.2.2 Importance:

Almost all the nations in the world follow the building codes documented within the nation. However, there some of the countries which purely rely on these codes prepared as well. Code is a set of commandment followed as per norms of government of the region. Those experts documented these through their experience and researches after reviewing the drawback for over a decade. The design codes of each nation primarily influenced by the climatic, topographic, and geological conditions persisting in the area wherein the structure exposed. Hence the design code has become a mandatory part in the constructional sector, and the country insists on following the design code. A building code formed due to life loss and the property because of the improper design & constructional practices. Now that the design codes formed the critical issues are almost nonexistent and uniformity is also maintained. Multiple uses of the design codes are sometimes chosen for a building to perceive the most cost-effective and adequate design by doing so. As a primary structural member of any building is its columns, transferring loads to the ground is through these members. The column cannot lay over the ground; therefore, a connecting element between the foundation and the steel column is the steel baseplate. Thus the steel column base plate is the critical element of any structure the purpose of it is to transfer the load evenly to the area foundation without failure. Hence to attain this task, suited building design needs to follow, which would guide the engineer to anticipate and make the required base plate connection without failure. Since each design code used would give a varied output, which means the size of the member would vary till it shown as safe so the base plate would also vary accordingly. Thus making the International building design codes crucial in the design of any element in a structural building maintaining maximum restriction in switching the between the codes.

2.2.3 Review of Literature study on the steel column base plates with Steel Building Design Codes:

The study of the steel column baseplate connection and the journals available is used to augment this study. However, the final task is to conclude which code would provide or recommend economic guidance based on minimum requirements, strength, design criteria. From the study conducted by Kameshki (Kamenshki, 1998) on steel beam, which is laterally supported & unsupported, column loaded concentrically, and beam-column subjected to only imposed and dead gravity loads. Designed concerning two codes namely BS 5950 and AISC LRFD about their respective yields which then he concluded that the laterally supported beam design was confirming to be more economical with the BS 5950 whereas the unsupported one to the AISC. Although for the column which was concentrically loaded the more promised design code was AISC alone. However, in general, he stated that AISC-LRFD showed more promising values for the economical design than the BS5950. The design codes form North America namely the American (AISC), Canadian (CSA), Mexican (RCDF) compared by Galambos (Theodore V. Galambos, 1999) concerning the stability of the design of the plates, beam-columns, column and beam. He concluded that fundamental concepts and the background of the research for all the codes compared were shown to be the same. The criteria noted about the strength were either similar or identical according to his study. However, he stated that there other significant differences like plate slenderness, shear capacity of webs.

In another comparative study between design codes carried out by Mourad (Mourad M. Bakhoum, Sherif A. Mourad, Maha M. Hassan, 2015) on the provision for loads, strength of sections in compressive loads as well as the flexural where studied when there is mixing of codes. The researchers established that choosing the loads from one code and the resistance for another to arrive at an economical result could lead to an unsafe design. Switching or mixing between codes would tip to an unconservative or conservative output as per the desired requirements like the section modulus, dimensions. However, this practice, as per the study carried out by the stated researchers, is not recommended. On the other hand, a comparative study about the wind effects on buildings carried out by Kwon (Ahsan Kareem, Dae Kun Kwon, 2017) complying with survivability design and the serviceability requirements in a crosswind and alongside directions. The wind effect on the column so as the base plate is crucial, so they stated that parameters so forth linked with the velocity of the wind subsidies in the differences generated through wind responses like the base RMS/peak acceleration and the moment /shear.

Further adding to this observation is that the ASCE had a distinct empirical expression concerning the reductive format, and interpretation was with more accurate data. An experimental study conducted by Stamatopoulos (G.N. Stamatopoulos, J.Ch. Ermopoulos,

2011) to describe the bending moment and the angle of rotation with their proposed formula. The output of the research carried out was satisfactory with a formula that was proposed and seemed to agree with the Eurocode as well. Another experiment study on the same conducted in Gheorghe Asachi Technical University (Silviu-Cristian Melenciuc, Andrei-Ionut Stefancu, and Mihai Budescu, 2011) with reverence to the flexibility of the base plate under brittle failure and the rotational capacity. The conclusion of the experiment was the increase in the stiffness of plate by giving it thicker and shorter would distress the joint rotation which the bolts through the holding the assembly down by the bolts deformation were in this phase should not be included or may be avoided at best in the seismic action. Another experiment conducted by Thambiratnam (David P. Thambiratnam, M. ASCE and P.Paramasivam) which had a column with an axial load and moments by loading' the assembly' eccentrically' were' in the parameters chosen was the baseplate' thick moreover, the load eccentricity. They concluded the load applied eccentrically had a higher impact on the strains generated than the thickness of the intended plate, and the failure of the base plate observed when loaded at high eccentricity by yielding. In the comparison of the BS and EC codes considering a multi-story braced steel frame down by Chan (Chee Han Chan, 2014) to claim the most economical design code. He stated at the conclusion that the design method with EC-3 had a reduced beam shear and moment capacity, meanwhile, structural columns had a compression capacity less than BS. He further narrated that the EC-3 design code had deflection reduced because of the fact of unfactored imposed loads. Thus EC-3 took arrived at more steel weight than the BS which proved to be uneconomical.

A study conducted by Johnson (R.P.Johnson, 2005) about the shear connection in the beams that supported composite slabs stated that the European code has not complete coverage on a specific problem and still needed development, unlike the BS code. A frame of two dimensional single story moment with a brace was experimented on by Celikag (Murude

Celikag, 2006) which subjects to second-order elastic analysis and loadings until failure. These compared with the two design codes, EC-3 and BS 5950. The results of this analysis were rather astounding that the steel frame designed with EC-3 performed better and were lighter than BS 5950. However, when the same frames subjected to maximum loadings, BS 5950 showed more capability wherein only gravity, dead and live loads as per the codes were considered. This shows research related to steel column base plate and effects due to the use of different design codes that have been going on since years with a review on its similarities and differences even though researches supporting were found to be a limited number.

CHAPTER 3: GENERAL COMPARISON OF EUROPEAN, AMERICAN, AND BRITISH STEEL BUILDING DESIGN CODES

3.1 General:

A general illustration of the foremost steel design codes chosen focused in this chapter basing on it's used quantities like actions, units, and strengths, safety factors which are the parameters. The narrative extends to the Ultimate Limit State and Serviceability limit state as well. This chapter intends to discuss the general comparison accordingly with the behavior of a steel column baseplate concerning each design code.

3.2 The Unit system:

The unit system followed by the most international codes is the Meter-Kilo-Second (MKS) which otherwise known as the metric system. The MKS includes the European, British codes but not the American code as it follows the United States customary units, which are the Foot-Pound-Second (FPS). It is recommended to design the structure in the respective code unit system to maintain the integrity and the easy understanding of calculations to avoid any error that might occur during the process. However, the majority of the world relates to the international metric system; the following chapters would be carried out concerning the metric system unless otherwise noted.

3.3 Actions/ Loads:

As discussed in the earlier chapters, even though the design philosophies remain the same, the loads and strength of a particular section and the safety factors would differ with each code taken for design. As the design area of the steel, the column is less due to the fact of it having a strength higher of compression and bending whereas the concrete which is going to bear the load comes from the column has less bearing strength hence there is a requirement of a larger concrete area. It reflects the flexural rupture of a puncture due to shear if a concrete section below column is significantly thin like a slab or thicker section like a pedestal so the concrete might fail in crushing if designing not done accordingly/properly. Hence to overcome this effect, a column base plate provided between the column and the concrete base. It helps the concentrated load from the column to the concrete to get distributed/dispersed to a larger area, and after that, it is transferred to the concrete foundation; hence, the system is safe. The loads which would be critical in the structural system for a column base are the axial, moment, and shear loads. As the self-weight of a structure is a permanent load taken to enable calculation of different materials with those unit weights specified by most codes fundamentally based on the units which are standard and hence would not differ much among the codes.

These loads would ultimately be transferred to the column base and then to the base plate creating the column reactions. These column reactions depend on these loads. Hence to summarize the course of action/loads on a structure in all the codes have the same philosophy, which is the:-

a) Permanent Loads: These loads include the weight as in the self-weight of the structure itself. It accounts for those nonstructural elements like the roof sheeting, ducts. Otherwise calculated from the actual weights of the elements.

b) Imposed loads: Those areas in a structure which are prone to those categorized as free actions in other words those loads which are doubtful to be permanent those loads are the imposed loads like the human load, table.

The following is a comparative table of the imposed loads between the three codes in use American, British, and the Eurocode. For convenience, the three types of buildings chosen are the Residential, office and shops, which are the majority types of structures build with their floor, stairs, corridors and the balconies.

IMPOSED LOADS FOR RESIDENTIAL PURPOSE (kN/m^2)			
	EUROCODE-1	ASCE -7	BS 6399-1
FLOORS	2.0	1.92	1.5
STAIRS	4.0	4.79	3.0
CORRIDORS	-	4.79	3.0
BALCONIES	4.0	2.87	3.0

IMPOSED LOADS FOR OFFICE PURPOSE (kN/m^2)			
	EUROCODE-1	ASCE -7	BS 6399-1
FLOORS	3.0	2.4	2.5
STAIRS	3.0	4.79	4.0
CORRIDORS	-	4.79	4.0
BALCONIES	3.0	-	4.0

IMPOSED LOADS FOR GENERAL SHOPS PURPOSE (kN/m^2)			
	EUROCODE-1	ASCE -7	BS 6399-1
FLOORS	5.0	6.0	4.0
STAIRS	5.0	4.79	4.0
CORRIDORS	-	6.0	4.0
BALCONIES	5.0	-	-

TABLE 3.1: Comparison of the imposed loads provision of the international building design codes

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores

[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3

NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex.

NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5.0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

TABLE 3.2: Reference of imposed loads from Eurocode-1-Part-1

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lbs (kN)
Apartments (see residential)		
Access floor systems Office use Computer use	 50 (2.4) 100 (4.79)	 2000 (8.9) 2000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters Fixed seats (fastened to floor) Lobbies Movable seats Platforms (assembly) Stage floors	 60 (2.87) 100 (4.79) 100 (4.79) 100 (4.79) 150 (7.18)	
Balconies (exterior) On one- and two-family residences only, and not exceeding 100 ft.^{2} (9.3 m^2)	100 (4.79) 60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors First floor Other floors, same as occupancy served except as indicated	 100 (4.79)	
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof) Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwellings (see residential)		
Elevator machine room grating (on area of 4 in.^{2} (2580 mm^2))		300 (1.33)
Finish light floor plate construction (on area of 1 in.^{2} (645 mm^2))		200 (0.89)
Fire escapes On single-family dwellings only	100 (4.79) 40 (1.92)	
Fixed ladders		See Section 4.4
Garages (passenger vehicles only) Trucks and buses	40 (1.92) Note (2)	Note (1)

(*continued*)

Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100 (4.79)	2000 (8.90)
Offices	50 (2.40)	2000 (8.90)
Corridors above first floor	80 (3.83)	2000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79) Note (4)	
Roofs	See Sections 4.3 and 4.9	

(continued)

Stairs and exit-ways	100 (4.79)	Note (7)
One- and two-family residences only	40 (1.92)	
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)	
Heavy	250 (11.97)	
Stores		
Retail		
First floor	100 (4.79)	1000 (4.45)
Upper floors	75 (3.59)	1000 (4.45)
Wholesale, all floors	125 (6.00)	1000 (4.45)
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit-ways)	60 (2.87)	
Yards and terraces, pedestrians	100 (4.79)	

TABLE 3.3: Reference of imposed loads from ASCE-7

<table>
<tr><th>Type of activity/occupancy for part of the building or structure</th><th colspan="2">Examples of specific use</th><th>Uniformity distributed load kN/m^2</th><th>Concentrated load kN</th></tr>
<tr><td rowspan="8">A Domestic and residential activities
(Also see category C)</td><td colspan="2">All usages within self-contained dwelling units
Communal areas (including kitchens) in blocks of flats with limited use (See note 1) (For communal areas in other blocks of flats, see C3 and below)</td><td>1.5</td><td>1.4</td></tr>
<tr><td colspan="2">Bedrooms and dormitories except those in hotels and motels</td><td>1.5</td><td>1.8</td></tr>
<tr><td colspan="2">Bedrooms in hotels and motels
Hospital wards
Toilet areas</td><td>2.0</td><td>1.8</td></tr>
<tr><td colspan="2">Billiard rooms</td><td>2.0</td><td>2.7</td></tr>
<tr><td colspan="2">Communal kitchens except in flats covered by note 1</td><td>3.0</td><td>4.5</td></tr>
<tr><td rowspan="3">Balconies</td><td>Single dwelling units and communal areas in blocks of flats with limited use (See note 1)</td><td>1.5</td><td>1.4</td></tr>
<tr><td>Guest houses, residential clubs and communal areas in blocks of flats except as covered by note 1</td><td>Same as rooms to which they give access but with a minimum of 3.0</td><td>1.5/m run concentrated at the outer edge</td></tr>
<tr><td>Hotels and motels</td><td>Same as rooms to which they give access but with a minimum of 4.0</td><td>1.5/m run concentrated at the outer edge</td></tr>
<tr><td rowspan="15">B Offices and work areas not covered elsewhere</td><td colspan="2">Operating theatres, X-ray rooms, utility rooms</td><td>2.0</td><td>4.5</td></tr>
<tr><td colspan="2">Work rooms (light industrial) without storage</td><td>2.5</td><td>1.8</td></tr>
<tr><td colspan="2">Offices for general use</td><td>2.5</td><td>2.7</td></tr>
<tr><td colspan="2">Banking halls</td><td>3.0</td><td>2.7</td></tr>
<tr><td colspan="2">Kitchens, laundries, laboratories</td><td>3.0</td><td>4.5</td></tr>
<tr><td colspan="2">Rooms with mainframe computers or similar equipment</td><td>3.5</td><td>4.5</td></tr>
<tr><td colspan="2">Machinery halls, circulation spaces therein</td><td>4.0</td><td>4.5</td></tr>
<tr><td colspan="2">Projection rooms</td><td>5.0</td><td>To be determined for specific use</td></tr>
<tr><td colspan="2">Factories, workshops and similar buildings (general industrial)</td><td>5.0</td><td>4.5</td></tr>
<tr><td colspan="2">Foundries</td><td>20.0</td><td>To be determined for specific use</td></tr>
<tr><td colspan="2">Catwalks</td><td>—</td><td>1.0 at 1 m centres</td></tr>
<tr><td colspan="2">Balconies</td><td>Same as rooms to which they give access but with a minimum of 4.0</td><td>1.5/m run concentrated at the outer edge</td></tr>
<tr><td colspan="2">Fly galleries</td><td>4.5 kN/m run distributed uniformly over width</td><td>—</td></tr>
<tr><td colspan="2">Ladders</td><td>—</td><td>1.5 rung load</td></tr>
</table>

Type of activity/occupancy for part of the building or structure	Examples of specific use		Uniformity distributed load kN/m²	Concentrated load kN
C Areas where people may congregate C1 Areas with tables	Public, institutional and communal dining rooms and lounges, cafes and restaurants (See note 2)		2.0	2.7
	Reading rooms with no book storage		2.5	4.5
	Classrooms		3.0	2.7
C2 Areas with fixed seats	Assembly areas with fixed seating (See note 3)		4.0	3.6
	Places of worship		3.0	2.7
C3 Areas without obstacles for moving people	Corridors, hallways, aisles, stairs, landings etc. in institutional type buildings (not subject to crowds or wheeled vehicles), hostels, guest houses, residential clubs, and communal areas in blocks of flats not covered by note 1. (For communal areas in blocks of flats covered by note 1, see A)	Corridors, hallways, aisles etc. (foot traffic only)	3.0	4.5
		Stairs and landings (foot traffic only)	3.0	4.0
	Corridors, hallways, aisles, stairs, landings, etc. in all other buildings including hotels and motels and institutional buildings	Corridors, hallways, aisles, etc. (foot traffic only)	4.0	4.5
		Corridors, hallways, aisles, etc., subject to wheeled vehicles, trolleys etc.	5.0	4.5
		Stairs and landings (foot traffic only)	4.0	4.0
	Industrial walkways (light duty)		3.0	4.5
	Industrial walkways (general duty)		5.0	4.5
	Industrial walkways (heavy duty)		7.5	4.5
	Museum floors and art galleries for exhibition purposes		4.0	4.5
	Balconies (except as specified in A)		Same as rooms to which they give access but with a minimum of 4.0	1.5/m run concentrated at the outer edge
	Fly galleries		4.5 kN/m run distributed uniformly over width	—
C4 Areas with possible physical activities (See clause 9)	Dance halls and studios, gymnasia, stages		5.0	3.6
	Drill halls and drill rooms		5.0	9.0
C5 Areas susceptible to overcrowding (See clause 9)	Assembly areas without fixed seating, concert halls, bars, places of worship and grandstands		5.0	3.6
	Stages in public assembly areas		7.5	4.5
D Shopping areas	Shop floors for the sale and display of merchandise		4.0	3.6

TABLE 3.4: Reference of imposed loads from BS5950-1

From the comparative table inferred above in the residential type, the most economical design code is shown to be British standards (BS 6399-1). Whereas in the office purpose building it is Eurocode (EC-1). Then again, from the comparative study of the shops, it is clear that British standards (BS 6399-1) are economical. However, to have a safer approach to the design of the structure, ASCE can be preferred but not recommended as it could overemphasize the required design. In an overall comparative study of the imposed loads from the international codes, it concludes that the British standards give those values which are more economical for the steel building design.

Wind load is another important in the series of loads, which is by default, considered in the load combinations of the design codes chosen for the work. The discussion on this load would restrict to a short generalization. The designing method for wind load action is almost similar in each code. In the country of U.A.E the necessary wind speed is taken to be 45m/s with 3s of gust effect which has considered as the fundamental in the AISC 360-16, BS 6399-2, and the EC-1-4 The design includes the combination of external and the internal pressure of the wind. The following is just a sample representation of the wind effect on the building.

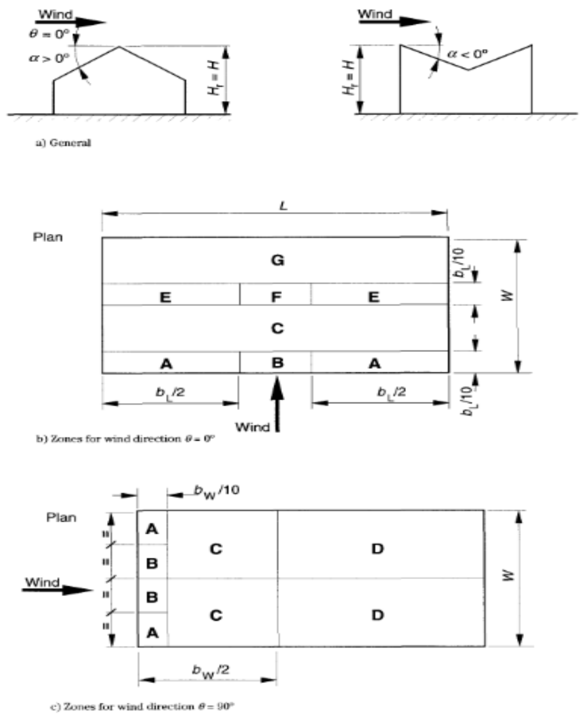


FIGURE 3.1: Example of wind effected on a roof of building as per BS 6399-2

The wind calculations are done for the critical angles, as shown in the above figure that is at 00 and 900. The calculations for exceptional cases like a parapet, tunnel. However the after getting all the pre-requisite inputs these calculations can be done accurately by the wind tunnel test which is recommended by most of the codes. Nevertheless, The wind load calculation can be understood more through the following chapter of the computer analysis of the structure. As for Seismic design as designed baseplate and the anchors should be compatible enough to achieve that required ductility of the frame for the columns that are the part of SLRS. The modification factor R is taken to be higher than 3 for the steel system, which is in the seismic load resisting system (SLRS) for buildings as well as for further structures. Sometimes shear forces from the earthquake are once in a while opposed by implanting the column section base and accommodating shear force into the floor framework. The column is to be provided with reinforcements to distribute the horizontal forces into the concrete.

The special requirements for the elements in the SLRS as per the AISC code need not apply since the strength required calculated at higher force levels. Braced type of frames with bases must intend for the essential strength of the components associated with the base. The segment-base association has to be planned not only for necessary compression and tension strengths of a section, yet in addition for the vital strength of braced type connection and bending resistance or fixing type for base as to those moments' that would arise' at the structure drifting of story' (inelastic floats as anticipated by code). Then again, the section base might intend for the amplified force got from applicable building code combinations of action/load, which also includes amplified seismic loads. Moment frame base can be planned to design as exact bases to pinned or in an accurate word as partial moment connections to be restrained or otherwise as a rigid, fully restrained moment connection. The goal of this issue of structure this association reliable with the reasonable conduct of a joint, representing a relative strain as well as stiffness ability of all components of the connection associated (baseplate, column, anchors, grout, and cement). Contingent upon the choice of connection, the section base should otherwise have sufficient strength to keep up the accepted level of fixity otherwise should have the capacity to give the anticipated shear strength wherein enabling the regular rotation to happen.

1. Rocking and rotation of the foundation may be an issue, especially the columns with the isolated footing.
2. The shear mechanism is not the same as amid the end plate of the beam and flange of a column as a mechanism between the column base and concrete or grout.
3. The standard diameter for holes for high strength bolts is different from the hole diameter of that of column base anchor rods.
4. There would be a straining of anchor rods which are installed in solid concrete more than that of high strength' type' of bolts' or where a beam-column weld type connection.
5. The column section end-base connection has significantly progressively load along longitudinal plane over flanges and less load in transverse when contrasted with the beam-column connection.
6. Column Section baseplates are bearing over concrete and grout, which are compressible even further than' a column' segment flanges of the beam-column connection.

3.4 Combinations of actions/loads:

Any structure always subjected to combinations of loads, and the course becomes critical to the safety of the building. Hence each code specifies the corresponding load combinations associated with the region of the design code. It is to check the structures' strength or any part of the building in combinations. The combination of action is the specific loads that the intended should multiply by the relevant partial factor specified in each code. Note that each code has a different approach to load combination because of the areas documented. These factored loads used in the most unfavorable realistic part for the consideration of the combination. When the combinations applied to the structure design, it evaluats that all the structural members present have to safe within its load carrying capacity. The following is the list of load combinations used for each international code chosen in this work. The development of the limit state design method about the probability base criteria led to the factored load combinations. Therefore each type of load is multiplication of the intended load, and the most critical load combination embraced. The format of the specific load with its partial safety factor is Design Load = Load X Partial Safety factor. The summarization of the load factors are the following

EUROCODE-0 (cl.6.4.3)
1.35DL+1.5LL
1.0DL+1.5WL
1.35DL+1.5LL+0.9WL
ASCE-7 (cl.2.3.2)
1.4DL
1.2DL+1.6LL+0.5l_r
1.2DL+1.6L_r+LL
1.2DL+1.6L_r+0.8WL
1.2DL+1.6WL+L+0.5L_r
1.2DL+LL
0.9DL+1.6WL
0.9DL
BS5950-1 (cl.2.4-TABLE 2)
1.4DL+1.6LL
1.4DL+1.4WL
1.2DL+1.2LL+1.2WL
NOTE: The load combinations includes only the dead, live and the wind loads as it is the mostly used combinations. DL = DEAD LOAD, LL = LIVE LOAD, L_r = ROOF LIVE LOAD, WL = WIND LOAD

TABLE 3.5: Comparison of the loads combination provision of the international building design codes

From the above table, it is evident that the American code chooses to have several combinations compare to the British and the Eurocode. As stated above, all the combinations of action for each code consider the effect of the wind load. However, the American code gives significance to the roof live load in particular. These factored loads are in fact to account for those imprecisions at the period of designing or at the ultimate design loads. The multiplication those intended loads with the partial factors specified by each design codes (Scott, Salgado & Kim, 2003). The following is the short comparison of partial safety factors assigned to each characteristic load at Ultimate Limit State.

LOAD	EUROCODE-0	ASCE -7	BS 5950-1
DEAD	1.35	1.2	1.4
LIVE	1.5	1.6	1.6

TABLE 3.6: Comparison of the Partial load factors as per international steel design codes

The above table infers that the partial factor applied of those to the dead load is high for the BS 5950 as and it simply overvalues the static cases of loads. It is to note the Eurocode gives the least value for the dead load as they described as the static load as well as the same with the American code. However, the live load factors chosen in both American and British codes are, in fact, 1.6. During a buildings service life, the variable loads require a factor of safety, which is more significant to consider the most critical load case of the design which would lead to an anomalous surge in the variable actions. However, the researches show having a more substantial value of the partial safety factor would lead to more significant design shear and moment forces, which then would result in the use of a bigger section. The sample calculation has been presented in APPENDIX A to show the variations in choosing the partial factor of each code. The following the inferences from the sample calculations as stated above the British code (BS 5950) yielded the highest ultimate factored load, whereas the Eurocode had the least value.

3.5 Classification of Materials and Sections:

During the design of any building structure, the materials integrated plays a vital role due to its effect on the cost, availability, strength, and integrity. It’s also crucial as to account those materials which are generally less expensive and wherever possible use thicker plates than detailing stiffener otherwise using other additions of reinforcements to accomplish equivalent strength with thinner plates to gain economy in the project. The type of material chosen for the steel design plays essential criteria as it governs the safety factor as well. The material chosen would decide the member carrying capacity of the given load in a structure. Each code has its material recommendation, but Eurocode and the British code suggest almost the same materials. The following is the gist of the materials from each code.

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40\text{ mm} < t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NLH	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

TABLE 3.7: Material Specifications as per EC-3

Steel grade	Thickness[a] less than or equal to mm	Design strength p_y N/mm^2
S 275	16	275
	40	265
	63	255
	80	245
	100	235
	150	225
S 355	16	355
	40	345
	63	335
	80	325
	100	315
	150	295
S 460	16	460
	40	440
	63	430
	80	410
	100	400

[a] For rolled sections, use the specified thickness of the thickest element of the cross-section.

TABLE 3.8: Material Specifications as per BS 5950-1

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53/A53M	B	35 (240)	60 (415)
ASTM A500/A500M (round)	B C	42 (290) 46 (315)	58 (400) 62 (425)
ASTM A500/A500M (rectangular)	B C	46 (315) 50 (345)	58 (400) 62 (425)
ASTM A501/A501M	A B	36 (250) 50 (345)	58 (400) 70 (485)
ASTM A618/A618M (round)	I and II ($t \leq w$, in. (MPa)) III	50 (345) 50 (345)	70 (485) 65 (450)
ASTM A847/A847M	–	50 (345)	70 (485)
CAN/CSA-G40.20/G40.21	350W	51 (350)	65 (450)
ASTM A1085/A1085M	–	50 (345)	65 (450)
ASTM A1065/A1065M	50 50W	50 (345) 50 (345)	60 (415) 70 (480)

TABLE 3.9: Material Specifications as per AISC 360-16

From the above tables, both European and the British follow the same type of material, but the correspondence of each material is to different thickness of the section size used. The EC-3 has two separate tables for both rolled and hollow sections whereas the British have a general table for the entire section sizes. The AISC code recommends the material grade as per ASTM standards, as shown in the above tables. However, there is no specification at what thickness the material used for the section sizes. In U.A.E the most common steel grade used is the S275 which the British material has a yield strength of 275 N/mm2, but in infrequent occasion American steel grade A6 used as well. It shows that material availability is abundant and economical for British standard materials. Note that all design codes mentioned in work have accounted for both yield and tensile strength for materials used. Like the recommended materials by the design codes they also recommend the sections as well namely

EUROCODE
European Flange Beams (HE)
European I Beams (IPE) & (IPN)
AISC
American Wide Flange Beams (W-Sections)
BS
Universal Beams (UB)
Universal Columns (UC)

TABLE 3.10: Sectional Specifications as per Steel design codes

Again the most preferred and the availability sections are the British sections in the U.A.E. as it is most economical, but in some cases, the American sections used as well. The sections mentioned above are the hot-rolled sections. A short study of the comparison of the section (E.S.Kamenshki et al., 1998) concludes that the AISC and the B.S. stipulate sections of four categories which are Plastic, Compact, Semi-compact, and Slender sections. The AISC compact sections used for the flexible design which overlaps with a plastic section in BS 5950, while partly semi-compact, slender sections and the compact sections correspond to that of the B.S.'s same parameters.

The below table shows that the limits of width thickness ratio is not worth the limits of the cross sections in AISC which are more generous than that of BS 5950 (E.S.Kamenshki et al., 1998).

Description of element (1)	Class of Section					
	AISC–LRFD compact (2)	BS 5950 plastic (3)	AISC–LRFD partially compact (4)	BS 5950 compact (5)	AISC–LRFD noncompact (6)	BS 5950 semicompact (7)
Outstanding element of compression flange	$b/t \le \frac{65}{\sqrt{F_y}}$	$b/T \le 8.5\varepsilon$	$\frac{65}{\sqrt{F_y}} < b/t < \frac{141}{F_y - 10}$	$b/T \le 9.5\varepsilon$	$b/t = \frac{141}{\sqrt{F_y - 10}}$	$b/T \le 15\varepsilon$
Flanges of I-section in pure compression	NA	—	$b/t < \frac{95}{\sqrt{F_y}}$	—	$b/t = \frac{95}{\sqrt{F_y}}$	—
Webs, where whole section is subjected to compression	NA	$d/t \le 39\varepsilon$	$b/t < \frac{253}{\sqrt{F_y}}$	$d/t \le 39\varepsilon$	$b/t = \frac{253}{\sqrt{F_y}}$	$d/t \le 39\varepsilon$
Web, with neutral axis at middepth	$h_c/t_w \le \frac{640}{\sqrt{F_y}}$	$d/t \le 79\varepsilon$	$\frac{640}{\sqrt{F_y}} < h_c/t_w < \frac{970}{\sqrt{F_y}}$	$d/t \le 98\varepsilon$	$h_c/t_w = \frac{970}{\sqrt{F_y}}$	$d/t \le 120\varepsilon$
Web, generally	for $P_u/\phi_b P_y > 0.125$ $h_c/t_w \le \frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}$ for $P_u/\phi_b P_y \le 0.125$ $h_c/t_w \le \frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right)$	$d/t \le \frac{79\varepsilon}{0.40 + 0.6\alpha}$	$\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}$ $< h_c/t_w < 970/\sqrt{F_y}$ $\frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right)$ $< h_c t_w < 970/\sqrt{F_y}$	$d/t \le \frac{98\varepsilon}{\alpha}$	$h_c/t_w = 970/\sqrt{F_y}$	when R is positive $d/t \le \frac{120\varepsilon}{1 + 15R}$ and $\left(\frac{41}{R} - 2\right)\varepsilon$ when R is negative $d/t \le \frac{120\varepsilon}{(1 + R)^2}$ and $\le 250\varepsilon$

Note: NA = not available.

TABLE 3.11: Comparison Of section classification between AISC and BS5950 (E.S.Kamenshki, 1998)

The following recommendation compared between two design codes of construction AISC and Eurocode -3. As recorded by ANSI/AISC 360-05 the following are the list of materials that can be' used for' steel baseplates' and anchor rods in the structures. The below table provides the lists of standard materials and a range of thickness available for the base plate for those materials.

MATERIALS FOR BASE PLATE AS PER EC-3				
STEEL GRADE	**$t \leq 40$ mm**		**40 mm $< t \leq 80$ mm**	
	f_y (N/mm^2)	**f_u (N/mm^2)**	**f_y (N/mm^2)**	**f_u (N/mm^2)**
EN 10025-2				
S275	275	430	255	410
S355	355	490	335	470
EN 10025-5				
S235 W	235	360	215	340
S355 W	355	490	335	490

MATERIALS FOR BASE PLATE AS PER AISC		
MATERIAL A.S.T.M	**THICKNESS (in.)**	**Yield Point (ksi)**
A36	Over 8	32
	¾ to 8	36
A572	¾ TO 6	42
	¾ TO 4	50
A588	> 5 to 8	42
	> 4 to 5	46
	¼ to 4	50

MATERIALS FOR BASE PLATE AS PER BS 5950-1		
Steel Grade	**Thickness Less than or Equal to**	**Design Strength p_y**
S275	16	275
	40	265
S355	16	355
	40	345

TABLE 3.12: Type of Materials for Base Plate

Below table provides the list of materials and diameter ranges for the anchor rods for those materials.

MATERIALS FOR ANCHOR ROD AS PER EC-3-PT1-8		
BOLT CLASS	**Yield strength $\int_{yb}$ (N/mm^2)**	**Ultimate tensile strength $\int_{ub}$ (N/mm^2)**
4.6	240	400
4.8	320	400
5.6	300	500
5.8	400	500
6.8	480	600
8.8	640	800
10.9	900	1000
Note: Nominal yield strength should not go beyond 640 N/mm^2 for those anchor bolts acting at a shear and not more than 900 N/mm^2 otherwise		

MATERIALS FOR ANCHOR ROD AS PER AISC		
MATERIAL A.S.T.M	**THICKNESS (in.)**	**TENSILE STRENGTH, Fu (ksi)**
A36	¾ to 4	58
A193 Gr B7	≤ 2½	100
	> 2½ to 4	115
	> 4 to 7	125
A307 (obsolete)	¼ to 4	58
A354 Gr BD	2½ to 4	140
	¼ to 2½	150
A449	1¾ to 3	90
	1 to 1½	105
	¼ to 1	120
F1554 Gr 36	¼ to 4	58
F1554 Gr 55	¼ to 4	75
F1554 Gr 105	¼ to 3	125

MATERIALS FOR ANCHOR ROD AS PER BS 6104-1	
BOLT CLASS	**Yield Stress (N/mm)**
3.6	180
4.6	240
4.8	320
5.6	300
5.8	400
6.8	480
8.8	640
10.9	900

TABLE 3.13: Type of Materials for Anchor Rods

The most extensive availability of the material for the use of the base plate is found to be A.S.T.M. -A36 unless' the accessibility' of approved or equivalent grade' is established' with the specification. However, when the column base is designed against large moments or uplift the material chosen to be is ASTM A572 grade 50, which tends to be more economical.

ASTMF1554 is a traditional material chosen for rods and concerning its availability, and grade 36 is the strength level opted to be used. Other use of grades should confirm before the specs. When large forces of tension caused by connections which have moment otherwise an overturning from uplift ASTMF1554 Grade 55 rods chosen. The use of Grade 105 ASTM F1554 chosen when it’s not possible to improve the expected amount of strength using the Grade36 or Grade55 rods as the Grade105 is a specially produced rod which is higher in strength. ASTMF1554 Grade36 ¾ in diameter rod material recommended whenever doable. Wherever additional strength is needed, take into account of rod dia. up to about 2 inches in ASTMF1554 Grade36 material afore changing to high-strength grade material.

ASTMF1554 Grade55 anchor rods with additional S1 requirements can order that would limit the carbon equivalent max of 45%, to provide weldability if needed. The supplementary is only needed when the welding is vital for fixes in the field. Using Grade36 gives an advantage of weldability without using the supplement. An additional supplement like Charpy V-Notch toughness are available for Grade 55 and 105; however, the anchor rods would have a default fracture toughness. The addition of such a supplement is expensive and won’t make a difference in the course of failure of anchors imperiled to fatigue type of loading. 95% of an anchor rod the fatigue life is deprived’ when the size of a crack’ is smaller/thinner than a few mm. Note that if anchor rods designed with additional redundancy, it would be more cost-effective rather than using/specifying supplemental CVN properties.

As we are dealing with metal corrosion is one of the major problems faced when it is exposed; hence, it requires the galvanizing of the anchor rods. There are two recommended process for galvanizing which are allowed in ASTM F1554:-

- ASTM153-Hot-dip galvanizing process
- ASTMB695-Mechanical galvanizing process

Note that all the threaded components of the fastener assembly should be galvanized using the same process if not it would pose or result in an unworkable assembly. So it is recommended to purchase the galvanized nuts and anchor rods preassembled and from the same supplier as well. Special kind of lubrication may be required as the assembly increases friction between nuts as well as rods due to galvanizing although the nuts tapped over.

3.6 General Behavior of the Assembly as per EC-3 Design Code:

The compressive point loads generally supported by a compressive member who is the column. Hence the compression check is critical of its member capacity. Effective length, Slenderness, and the compression resistance are the ones to decide this capacity of the steel column.

Effective length (L.E.) determined from the length from center-center of restraints with those of restraining members. Depending on the restraint conditions, the member which carries more than 90% reduced plastic moment capacity M¬r within in the incidence of an axial force, and it presumes that the member is unable of providing directional restraints.

The Slenderness of the column or the compression section is the effective length to the radius of gyration wherein this concept does not support angles, t-sections. Columns that subjected to a combined compression and moment forces fact governed by the section capacity and the member buckling resistance.

The assembly of the steel column base plate if assigned for two different stages that are for the resistance and the stiffness. The assembly for the resistance of the column base plate is on the equilibrium of plastic force (Wald et al., 2008). As per the effect of the action/load combination, there three patterns formed which are :

- The condition wherein there is no tension in the anchor bolts, which is because of standard and high force load. The concrete collapse appears even before the stress develops in the part of the tension.
- The scenario in which only one anchor bolt row is in tension which occurs due to the small loaded by regular forces compared to that of concrete's ultimate bearing capacity. Due to the yielding of bolts or plastic mechanism occurred in base plate break down procedes even before the stress of concrete bearing reached.
- Another part is tension effecting in both rows of anchor bolts when the assembly loaded by normal tensile force. The guidance by bolt yielding or the base plate plastic mechanism holds the stiffness. A pattern of such type occurs mostly in those base plates which designed for forces of tensile and the contact between concrete and baseplate intended by doing so.

The connection subjected to axial and the moment force as like in any other code criteria. The neutral axis position calculated as per the resistance produced in the part of the tension. Then the determination of the bending resistance is by a hypothetical scenario of the internal force distribution by the plastic distribution. The effective area considered, which is under the steel base plate, has been taken as the active section of an equivalent rigid plate, which is corresponding to the calculate of an equivalent T-stub with effective width. The assumption of the compression force to act at the center and the tensile force located in the middle or at the edge depending upon the number of the anchor bolts (Thambiratnam, Paramasivam, 1986). Note that at the Ultimate limit state (U.L.S.), a load of failure is critical and under the service load the concretes failure, and the elasticity checked.

The resistance assembly of the steel column base plate behavior differs from the normal base plate. These type of the base plate are compression or the tensile resistance of its apparatuses. The other elements in the mechanism are the anchor plate in tension and of course, the bending. The process to calculate the resistance in forces and the bending moment would follow the same as in all other connections. Primarily the component resistance is calculated. The active areas in contact underplate and base evaluated with those internal force equilibrium for part of tensile resistance. Thus from a contact area that identifies is evaluated with a resistance of bending and lever arm base of the column for a specific acting force which is a similar procedure to that of a column without an anchor plate is.

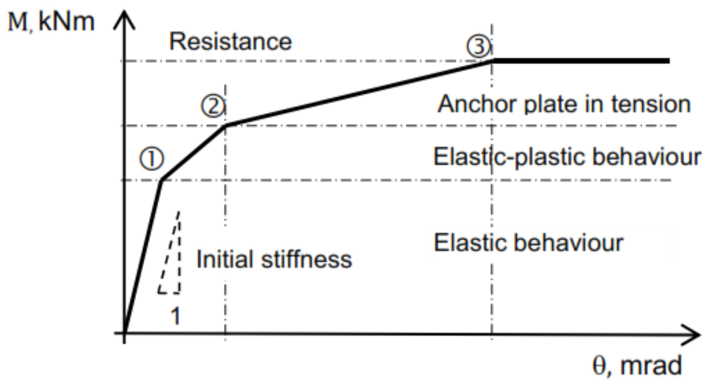


FIGURE 3.2: Moment Rotational Diagram of Column Base with Anchor Plate(Research Fund for Coal & Steel, 2014)

The assembly for stiffness preferably has a different approach even it little similar to that of the resistance base plate. As per the calculation (Wald et al., 2008) is attuned beam to column calculation for stiffness as well the only difference occurs in the two procedures is the connection of base plate to the column has added. In this procedure of the stiffness calculation, only the effective area accounted. The compression forces located in the middle of the compression area and at the anchor bolts the tensile forces located. The determination of the rotational bending stiffness evaluated during the constant eccentricity at proportional loading. As per the eccentricity by the activation of the anchor bolt, three collapses are estimated as per

(Wald et al., 2008) which are when the tension effected to one row of the anchor bolts is when loaded with small normal force compared to that of the concrete capacity of the ultimate bearing. Like the resistance plate, concrete bearing stress is not touched for the collapse to happen, but it happens due to bolts' yielding because of the mechanism of plasticity in the baseplate. The second scenario occurs is when the anchor bolts encounter with no tension where it develops during the stage of high loading of the normal forces. The concrete witnessed even before the stresses developed in the part of the tension. The procedure of calculations for the stiffens for an open section of I or the H section, and hollow sections can be calculated assuming two webs into account. The anchor plate for stiffness however assembled form the component's stiffness deformation.

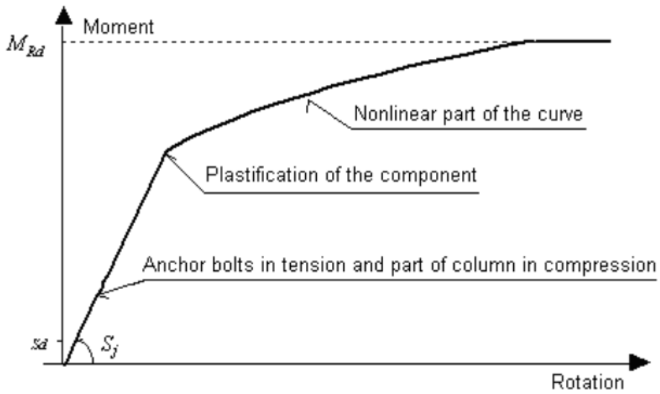


FIGURE 3.3: Moment Rotational Curve for Proportional Loading (Research Fund for Coal & Steel, 2014)

3.7 General Behavior of the Assembly as per AISC Design Code:

The procedure selected for the design of the base plate is done as per column reactions, as discussed in previous sections of work. It is to be noted that in American design of the connection of base plate the concrete pull-out is calculated as well when compared to the other codes. Hence before discussing in detail this section of the chapter would cover the general idea of the steel column base plate as per the AISC design. There are 3 sorts of loading in which the base plate is effected and by reckoning.

Note within the 1st condition, and typically within the 2nd condition, the baseplate is beneath compressive pressure over its complete area after which it happens the minimal anchorage would be enough to maintain the position of the baseplate. However, this also isn't crucial. If the case is where there is a large eccentricity of loading leading to a comparatively large moment, then it results in the lifting of the base plate. Henceforth during this case, the anchor bolts are going to be necessary to take care of equilibrium. A layer of grout is often most well-liked beneath the plate, which ensures an even and effective distribution of load.

Development of compressive stresses in concrete which is ascertained to be little at plate perimeters and that largest below the column. The distribution of a no uniformity within stresses of bearing is because of baseplate being, and it undergoes bending. Magnitude relation of the realm of concrete to it of the plate and also the depth of that of a concrete' foundation' was imminent' to own influence over an axially loaded base plate's behavior. Those baseplates below the eccentric' loads' of action or haven't received a lot of' consideration. The author Salmon, et al. established a technique for anchorage's higher and ultimate' moments at the lower bounds where evaluated with constant load in' axial. Their ways supported by the analytical approach about moment rotation of column anchorage characteristic, so an Avery universal testing utilized for the test

To proportion and analyze base plates beneath moments and axial loads, 2 strategic methods prescribed, which are the ultimate strength and working stress methods. With the former methodology, the behavior of the base plate relies on service loads or the design, whereas the latter methodology relies on the behavior at ultimate loads which are assumed. Sometimes the working stress is employed to detail and design base plate and also the method of ultimate strength to be employed to ascertain the design therefore obtained.

The observations from the analysis it was that all the baseplate used had the first mode of failure to be the concrete cracking even though the eccentricity was at the lowest whereas the primary failure for those base plates subjected to higher eccentricities where the base plate yielding but if the base plate where thicker, the case would have been the anchor bolt failure in tension and yielding. However, in the case of thin baseplates, the failure occurs conspicuously because of the relative flexibility of the plates.

In most cases, elemental cracks within concrete began' at the highest layer' wherever the plate connected towards the concrete. As the applied load increased, then these cracks unfold to the lowest of the concrete. So it was inevitable that at the larger eccentricity anchor bolt fails for, the thicker base plates and collapse occur from a thinner base plate. Note that as the thickness of the plate is increased the strain reduces.

Thus the equation formulated to equate the eccentricity under a certain load is

$e = M/P$

where e = eccentricities

M = the max. moment produced

P = applied load

The anchor bolts were therefore designed to avoid the anchor bolt yielding be the primary failure.

Failing occurred at very low eccentricity for the plates from the concrete cracking. The first mode of failure in all alternative scenarios was from baseplate' yielding. The yielding' came about at the column baseplate junction on constant aspect because of the load. It's not the thickets baseplate which would sustain the biggest moments all the scenarios. Inbound instances, the thicker base plate behaves sort of a rigid plate and can incline' to ride abreast of an edge which in turn would cause premature collapse and the bearing stress to be large.

Note that there is an increase in strain along with eccentricity at a constant load as well as strain reduces when the thickness of baseplate increases so to have conclusive results larger base plate variation in thickness accounted. Throughout associate degree experiment the existence confirmation strain' profiles and contours' of a vital section of most compressive strain, a section of zero strain, and the vicinity of tensile' strain' on the base plate top side.

It seems that base plates that are flexible when loaded at high eccentricities would fail due to base plate yielding and also has to consider that it will not continually behave as expected strategies deliberately.

3.8 General Behavior of the Assembly as per BS 5950-1 Design Code:

The steel column behavior is almost similar, as described in Eurocode-3. The column is a compressive member in the structure wherein the buckling length, slenderness, compressive resistance, and the buckling resistance governs the members capacity to carry the applied compressive force on it.

In the majority of the cases, the loading in a building column is designed to designate only for compression through axial with a small amount or maybe no. Uplift, either a combination of shear and tension or just shear. However, there is some application where there is pure tension which commonly called as pull-out. Shear can be transferred by the friction against the concrete or grout pad if the base plate remains in compression; hence, the designing of the anchor rods is not necessary for shear. Either by embedding or shear lug addition below the baseplate can also facilitate to resist large shear forces by bearing against concrete. Moment connection in the column baseplate can be utilized to repel seismic and wind-loads on the structure.

Development of force couple between tension in few otherwise all of the anchors and bearing on the concrete can be used to resist the column base moment.

Due to subjected creep effect over anchors expanding means that their behavior concerning the sustained pure tensile load calls out for high factor of safety to be applied. To obtain a safe working load, a factor of safety(f.o.s) of 3.5 is applied or 2 to get the design capacity for factored load design. When required, the column baseplate connection is also competent in transferring forces causing uplift and can transfer shear using its anchor rods — however, the technical data from the manufacturers based on the pull-out tests.

The load capacity depended on the following:-

- Use of slotted holes
- Whether the applied load is static or cyclic load
- The edge distance of the base material
- Condition and the strength of the base material
- Length of embedment
- Fixing spacing when in a group

It is recommended to verify the availability of materials for both anchor rods and base plate with the steel fabricator before confirming with the design of the column base assembly.

Generally, the classification of the column bases is of two types, which are: embedded bases and exposed bases. There is not much difference between the two of the types as they contain the same components. Each component of the column base has to be designed to perform its function concerning its requirement, fabrication, erection and moreover the forces it should transfer from the building to the foundation with utmost efficiency and safety.

A typical column base is designed to transfer the compressive axial load whereas a braced frame column base is designed to perform to transfer horizontal shear or if required tensile axial loads when subjected. Moreover, moment frame columns base aimed for an arrangement of compression through the axial, shear horizontal load as well as the moment.

In the case of compression by axial, the force is dispersed from column to the baseplate by direct bearing, whereas during tension by axial, the tensile forces directed towards anchor rods from the base plate. The column base exposed to the moment distributes the force couple to the foundation from the column through a combination of bearing and tension in the base plate and anchor rods respectively. For those column bases with shear which resists amid foundation and baseplate where the mechanism to be chosen carefully by the respective designer. When the column base subjected to uplift which transfers through baseplate bending, when uplift results in large tension forces, stiffeners are attached to the column. This ensures that forces from flanges of the column to be transferred to anchors straight by evading bending of the baseplate.

Holes provided for anchors in the baseplate are large, which tends to the difficulty of ensuring the transfer of shear forces. If the column base is to be designed to carry the shear forces, the design should account for the baseplate for its effect on the size of holes. Often achieved with a standard hole with a washer. In this case, it should distinguish that rods' (anchor) designed for bending between the concrete embedment and the washer. A smaller of say 26mm diameter with the base plate of less than 32mm thick and a 20mm in rod diameter can be designed criterion for a column subjected to only axial compression.

Through friction the shear forces can be transferred only if the shear force is relatively small, this is due to the dependable shear friction or the axial compression load between the surface of the foundation or grout and the steel base plate. If the shear force is large the transfer may require through embedment of a column base or by shear lug utilization. Moreover, hairpin reinforcing bars and tie rods can be used for the transfer of large shear forces if it may require from the column end base to the concrete foundation. However, the usage of anchors for the transfer of shear advised if it’s necessary to use it with caution. It is vital to design the column baseplate type of connection for its strength expectancy and recognize the connections that affect the behavior of the structure as well.

CHAPTER 4: PARAMETRIC REVIEW ON THE STEEL BUILDING DESIGN CODE PROVISIONS

In this chapter, the parametric review with the relevance of each code to the steel column base plate shall be discussed to understand through which a better design is possible.

4.1 General Concept of Design:

A column base plate connection is a bolted end plate but to the bottom of the column and the connection is mainly designed for the transfer of the axial force and the moment between the concrete and the steel members of the structure. This connection has certain exceptional features that are different from a typical end plate connection.

- In the column base plate connection, the forces considered more critical would be the axial forces.
- It is the strength of the packing of grout or mortar available decides the strength needed to withstand for the compression force dispersed from steel to concrete over the area of contact between them.
- The transfer of tension from the structure is by restricting bolts by holding it down and is anchored to the concrete substructure.
- Under tension for concrete cannot be relied on to produce prying forces, unlike the steel-steel contact endplate type of connections. So bending in single curvature should be considered for the steel base plate.

It seems that an unstiffened base plate tends to be thicker in comparing with the beam-column endplate connection. According to the moment direction, the proportional specifics, but in some cases, disproportionate details were chosen as well. The requirement of connection is usually to transfer the horizontal shear with the help of bolts or via friction. However, it is not mandatory that the horizontal shear forces are distributed equally to all the bolts in the assembly unless there is an inclusion bolt with washer plates welded over it in the final positions. As mentioned before in this code, shear studies welded beneath the base plate to counter heavy horizontal shear forces. Aforementioned the grouting in almost all the scenarios prove to be significant and necessary and requires exceptional attention.

The bolts that used to hold down are square in cross-section; hence, the washer plates used are square-cut to prevent the bolts from turning unintentionally. Suppose the bolts are not shaped then a keep strip is used to make sure the bolt won't turn by welding it to the bottom of the washer plate which would be contiguous to the head of the bolt.

It is critical that the anchor bolts need to designed in synchronization with reinforcements in the base when large forces and moments are expected. A 20mm to 40mm dimension of the thickness for bedding material is typically preferred so it gives a rational contact for bolt sleeves grouting which is necessary for the corrosion prevention and to ensure a thorough filling of the space between the concrete and the base plate and it also ensures the tolerance for the leveling.

As the base connection is the most rigid connection relative to any other connections in the structure as it affects the overall performance; hence, it has a more considerable significance. It is noted that even the unstiffened base plate is proven to be considerably stiffer than a usual end plate connection. It is due to pre-compression and thickness of the column base plate which contributes to this situation, but no connection is as stiff as concrete, in turn, the soil in which transfer of moment accomplished. There are other situations like the tendency to creep beneath a sustained load as well. The base connection is not rigid until the concrete the base join itself as relatively stiffer; this often can evident by inspection. Typically the strength of the grout is chosen equal to that of the concrete beneath that is the base which the material used can be the fine concrete, mortar or one of those branded non-shrink grouts. The placement of the high strength grout should be the ultimate unique control and caution to avoid any air bubbles and voids, so forth. If such an exceptional technique of controls is not available a design strength, 15N/mm2 limit suggested regardless of that of the concrete grade.

4.2 Overview

4.2.1 Steel Building Design Code EC-3:

The design procedure is usually on the method of trial and error through which the selection of the base plate and the anchor bolt. The factors by which these selections depended on are the moment and the axial forces. The following are the gist of the for the resistance evaluation:

1. The axial force affecting an equivalent T-stubs for both flanges identified. The forces are to be anticipated concentric with the flange if it is in compression whereas in tension the forces are presumed to be along the anchor bolts line.
2. Compute the T-stubs resistance in compression and then in tension
3. Then the shear resistance if confirmed for the connection involved
4. After that, the anchorage of the bolts is verified.

The resistance in a T-stub base plate provided by tension caused in holding down bolt outside of a column flange and cause of compression due to effect in the zone of concrete which his concentric with flanges of a column. But the model is limited by the bending-moment which can be huge or small which would be in relation with the axial force. The compression resistance is not accounted under the web if the moment is small with no existing tension. It can be surpassed by the force evaluation in web and the flanges which is then compared with the resistance available. Whereas if the value is large for a moment, then the possibility of larger resistance of moment is ignored for a reason that zone of compression which is outside of the column. The selection of eccentric compression zone surpasses this scenario if the eccentric design intended for the T-stub.

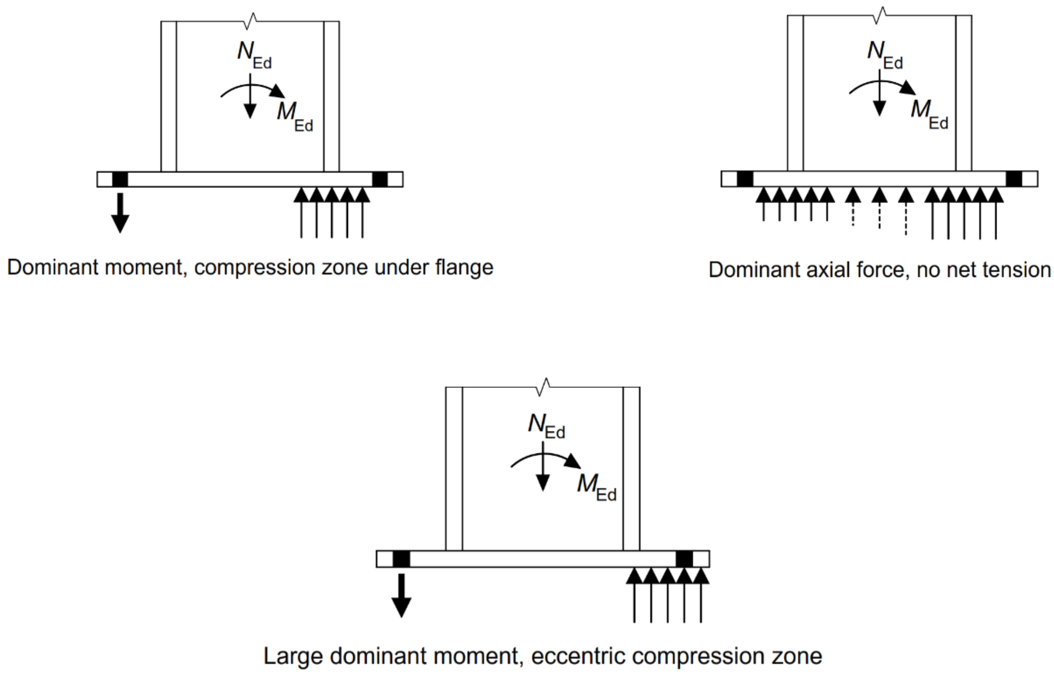


FIGURE 4.1: Range of design situations as per EC-3

The compressive reactions are assumed to be repelled centrally underneath a flange of the column which is at a distance of zc. The distance zc measured from the centerline of the column on either side. There is a possibility of attaining the dimension of zc to be higher if there use of an eccentric compression zone.

4.2.2 Steel Building Design Code AISC:

This chapter provides a design outline explicitly using the AISC code, which provides design for typical column base plate connection in structural buildings. A direct approach of LRFD or A.S.D. load combination done for the column base end and anchorage design portions. However, the embedment of anchor rods into the concrete is not readily designed by A.S.D. because of which LRFD exclusively based on the strength approach for such kind of embedment design. Whereas other components in the foundation assembly containing baseplate and sizing of anchors are likewise proficient for the evaluation by using load methods either LRFD or A.S.D.

The designs contemplated for the 5 load case scenario considered, which are:

1. Axial loads
2. Base plate with shear
3. Base plates with moments

For the moment and the shear designs are frequently executed freely for the column base connection which seeks to accept that there is no significant association between them.

The general conduct and force distribution for a base plate association with anchors would be elastic' until and unless the column section formed with a plastic' hinge, a plastic system frames in the' baseplate, the concrete' in bearing pulverizes, due to tension causing the yield of anchors, or the strength of pullout in concrete of the anchors assembly. If the concrete pullout' strength of anchors assemble is more significant than most minimal of former previously mentioned limit expresses, the conduct, for the most part, would be ductile. In any case, it isn't always essential or conceivable for a foundations design that would avoid cracking/failure of the concrete. For instance, in a structure that is loaded statically, if strength is a lot bigger, then ductility isn't essential, and it's adequate to design condition with a limit of shear or tensile quality of the group of the anchor rods overseeing the structure design. In any case, outlines intended for S.L.R. are required to act in a ductile way and, for this situation, it might be necessary for the design of the foundations and the column segment baseplate with the goal that as far as possible conditions of shear or tensile quality strength of the anchors amass don't administer the design.

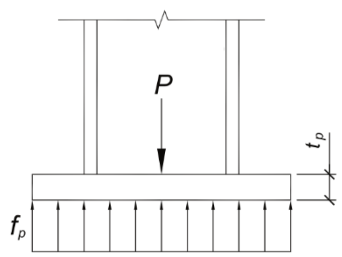


FIGURE 4.2: Load Distribution Pattern (AISC)

4.2.3 Steel Building Design Code BS 5950:

The method used in the studying of a' steel column' end baseplate in this design code is called the effective area method. The steel column bases should be competent enough in its size, strength, and stiffens to transmit the bending moments, axial and shear forces to their respective foundations without compromising the member load-carrying capacity. The inclusion of the anchor bolts or the holding down bolts should be preferred and provided when necessary. As per the uniform distribution of pressure between the baseplate and the support determines the nominal bearing pressure, for concrete support as in the concrete foundation the criteria is that the less quantity among its characteristics cube strength or the bedding material. The method used for the steel base plate in BS 5950 is the effective area method. A portion of the steel base plate area is taken as ineffective when the size of it is larger than the required amount to frontier the nominal bearing pressure.

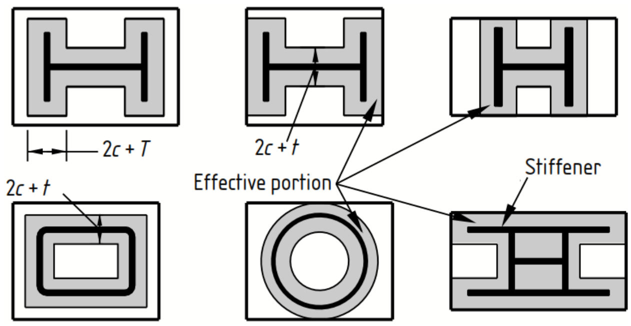


FIGURE 4.3: Range of effective areas in different sections (BS 5950-1)

4.3 Parametric Aspect:

4.3.1 Shear:

4.3.1.1 EC-3:

The shear is transmitted through the base plate to concrete in four ways, essential which are:

- The total compression presumed by a resistance of 0.3 times, which is by friction.

- Through bearing shafts of bolts and plate as well as between concrete and bolts which is surrounding them in bearing.
- Fixing tie bars directly.
- Positioning the base plates directly packed with concrete in a shallow pocket.
- or by welding the shear key over the beneath of the steel column base plate.

Friction alone is sufficient to define the transmission of shear in a section in a basic form possible, if not the most typical practice is to assume that the shear is getting transmitted through the bolts which are holding down when friction is not sufficient enough.

Things To Be Considered While Designing For Shear:

- The shear isn't shared equally among the bolts present in the assembly. It is because few bolts in the connection may not be in full contact with the connection plate as the bolts are in the clearance hole. The simple way to overcome this problem is to assume that the all the bolts in the connection are the effective but more efficient way is to weld a washer plate with a hole that would fit with connection precisely thus ensuring all the bolts are having an equal distribution of load and are in bearing.
- In a foundation the bolt position is to have a careful consideration as the resistance of the bolt tends to decrease close to the edge.
- The shear applied to the plate is above the concrete level so the bolts may subject to bending for example if the grout is not properly placed; hence, there is a decrease in the resistance.
- The assumption of resistance in bolts is because it is cast solidly into the concrete. For this assumption, it is mandatory to have a certain level of practice of the placement, control, and care for the grout to work in full capacity.

- If through friction or by the holding down bolts the transfer of shear force is not possible then there few ways in which the goal achieved in the following methods
- The column embedded in the foundation or shear-stub is joined towards underneath the baseplate by welding.
- By connecting the concrete floor slab and the column by introducing a tie bar or similar element or by casting slab surrounding the column.

The selected bolts are subjected to verification for the bearing in concrete as well as on the plate. The resistance of shear guaranteed if the bolts are cast into concrete solidly. It could be that the design established an effective length of the bearing of concrete (3d) and 2fcd as the average bearing stress wherein the fcd is concrete's design strength by compressive of foundation or grouting considering the weakest. If this method proceeds, then the selected bolts should fence with reinforcements. Note that in the direction loading if the edge distance of the bolt center from the concrete is less than 6d shouldn't be considered. The condition for evaluating the bolts subjected to combined tension and shear is

$F_{v,Ed}/ F_{v,Rd} + F_{t,Ed}/ 1.4\ F_{t,Rd} \leq 1.0$

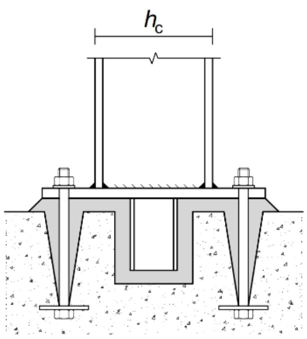


FIGURE 4.4: Detail Provision of Shear Stub (EC-3)

To choose a shear stub for the desired section, it's a standard practice to follow approx — 0.4 times to that of depth the column section for an I-section shear stubs depth. The effective depth is not to be more than 1.5 times of the stub's depth and should be greater than 60mm. The slenderness of the flange for an I-section is to be limited to concur with $b_n/t_{fn} \leq 20$.

The transmission of the load assumed is through the vertical faces of the stub in bearing. To authorize for any inconsistencies in that zone, distribution through the triangular method is assumed, and the small grouting space disregarded. 'fcd' is taken the max bearing stress and the design strength' by compressive' is taken the weakest value of the grout, and the concrete which the resistance subjected formulated as

$V_{Rd} = b_s d_{eff} fcd$ (For I or H section)

A secondary moment is encountered when there is an eccentricity between the reaction caused horizontally and applied shear over a stub which is M¬sec,Ed which assumed repelled through a combined force of compression beneath a flange as well as the shear stub being concentric with tension.

$M_{sec,Ed} = V_{Ed}(h_g + d_{eff}/3)$

$N_{sec,Ed} = M_{sec,Ed}/(h_s - t_{fs})$

where $N_{sec,Ed}$ =is the applied forces in stub flange

The flange resistance is given by

$b_s\ t_{fs}\ f_{ys}/\Upsilon_{M0}$

Weld among the flanges of stub and baseplate designed as weld which would be transverse has to be considered with those forces of the design 'Nsec,Ed' whereas for weld for a' web of the stub and the base plate is to be considered weld in the longitudinal direction for the intended shear force design 'VEd'. Verification is to be done whether the web of the column is subjected to the concentrated force from the flanges of the stub by considering the effective breadth 'beff'

$b_{eff} = t_{fs} + 2s + 5t_p$

where t_p = is the desired thickness of the baseplate

s = length of the weld leg to that of the stub flanges

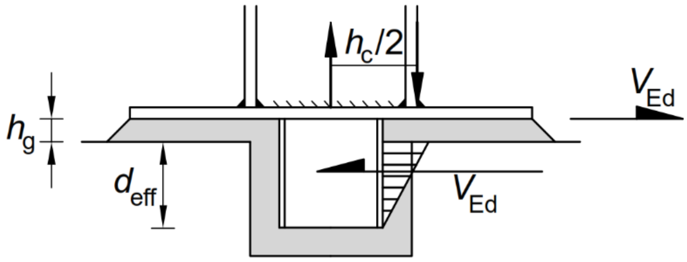


FIGURE 4.5: Detail Provision Secondary Moment (EC-3)

The shear resistance for a stub assuming the section is I-section is to be verified as

$V_{Rd} = A_{vs}f_{ys}/\Upsilon_{M0}\sqrt{3}$

where f_{ys} = the shear stub yield strength

A_{vs} = shear stub area

4.3.1.2 AISC:

The transferring of the shear to the concrete foundation through the governing of column base plate by 3 essential methods which are

- By shear through anchor rods
- By the use of friction between the concrete surface or the grout and the column base plate
- Using the bearing occurred through the base plate and column and use of shear lug against the concrete.

Friction

Usually, the development of the sufficient shear resistance to the lateral forces is through the compression forces between the concrete and base plate. The factored compressive loads (Pu) should comply with the lateral forces (Vu) evaluated where it results in the required shear contribution hence as per ACI the following is formulated to calculate the shear strength as

$\varphi V_n = \varphi \mu P_u \leq 0.2 f_c' A_c$

$\mu = 0.55$ (friction coefficient b/w steel and the grout)

$\mu = 0.7$ (friction coefficient b/w steel and the concrete)

Bearing

As mentioned in the above methods, shear can also be transmitted to foundation with utilization of shear lugs by bearing or the embedment of the column into concrete foundation.

If the use of shear lugs is intended then the use of a legitimate confinement with the bearing to transmit shear is recommended by ACI 349 as the shear is at first transmitted through the anchors over to the concrete/grout through bearing enlarged by shear obstruction from impacts of confinement related with those tension' anchors and outside simultaneous load by axial. Shear at that point advances into a mode of friction- shear.

The limit of bearing (φP_{ubrg}) prescribed as per ACI is $\varphi 1.3 f_c' A_1$. Utilizing $\varphi = 0.6$ reliable with ASCE 7 load elements, $\varphi P_{ubrg} \approx 0.80 fc' A_1$ = shear lugs area of embedment which excludes the column of the lug in touch with the grout. ACI suggests for those columns or base plates where it is embedded in adjacent with the surface of concrete against the bearing

$\varphi P_{ubrg} = 0.55 f_c' A_{brg}$

where A_{brg} = The region where the contact occur column and base plate over concrete.

As per ACI 349 due to confinement where it takes in the account of effect of loads acting external and tension anchors as well so the shear strength of the anchorage is taken as

$\varphi K_c (N_y - P_a)$

where P_a = The anchorage factored axial load (external) where it is –ve when there is compression and +ve when there is tension. When the case where P_a is –ve it is recommended to verify the existence of P_a in the occurring shear force.

$\varphi=0.75$, $K_c = 1.6$

$N_y = nA_{se}F_y$ = anchors (in tension) yield strength

So we can come to a conclusion by the following formula for lateral resistance as

$\varphi P_n = 0.55 f_c' A_{brg} + 1.2(N_y - P_a)$ (bearing on side of base plate or column)

$\varphi P_n = 0.80 f_c' A_l + 1.2(N_y - P_a)$ (shear lugs)

As per ACI 349 recommends to consider other than bearing failure for bearing of those column embedment's / shear lugs towards the direction of the concrete's free edge design of the concrete part of the shear lug for its shear strength shall confine with the uniform tensile stress which is $4\varphi\sqrt{f_c'}$ (where $\varphi=0.75$) where in the effective acting area of tension is determined by the 45^0 plane of projection towards free surface from the shear lug's bearing edge. While considering the projected area for the design it is be noted that the bearing area of column embedment / shear lug aren't to be included. The value to be used for 'φ' will be the limiting value to the capacity of shear of embedment of column or shear lug.

Bending of the base plate should be accounted for, which occurred due to the forces' prolonged in shear lugs. It can be due to unique concern' while the base' shears are huge maybe because of the bracing forces and due the force about the shear-lugs prevailing about the column's frail axis which causes bending. As a standard guideline, it is recommended to keep the thickness of both shear lug and the baseplate to be equal or the baseplate thickness to be more than that of shear lug thickness.

As per the provision of design allowed by the ACI 349 to accommodate multiple shear lugs which can be utilized so as to resist the effect of huge shear force. It's also important to account enough size for the grout pocket so as to ease the placement of grout where the grout should be a non-shrink and flowable material.

Anchor Rods which are Shear

Utilization to transmit the forces of shear via anchors must inspect cautiously because of a few assumptions are to be contemplated. Specific consideration must note to ways in which the force transmitted from the baseplate over to the anchors. Utilizing AISC-prescribed opening sizes of an anchor rod, slip which is considerable of the baseplate may happen before the baseplate endures in contradiction of the anchors. Impacts of this particular' slip assessed and note due to the employment tolerance, not the majority of the anchor rod would get a similar force.

While utilizing just two of the anchor rods for the shear transfer, it is recommended to have a cautious approach, except if different arrangements are made to level the load to all anchor rod. Equal transfer forces through lateral' can be done to all or specific' anchors, by utilizing a welded plate-washer in amidst baseplate and anchor rod nut. The holes on the plate washers should be larger than the intended anchor rod. Now for the case to equally transmit the shear to all anchors, a legitimate setting-plate thickness can be utilized and afterward field weld over to the baseplate after the erection of a column. It can't underline ample that problem arising in the period construction of the column bases have to account in consideration of the utilization of shear in design which encountered in the anchors.

Shear should be passed to the concrete as soon as the shear transmitted to the anchor rods. Washer plates are utilized to exchange shear over to rods, some anchor rods bending can be typical inside the thickness of baseplate. On the off chance that just two anchor rods utilized for shear, transmission occurs inside the baseplate, so rod bending disregarded. No' bending of the anchors inside the grouting reflected as per shear friction hypothesis. If the reverse curve bending expected, the moment in the anchor rods can be. A separation by half could be considered for the lever arm amidst the mid’ points of the direction of a washer-plates to the highest point of the surface of grouting. ACI 318 necessitates that the anchor limit increased by 0.8, where the built-up grout pad used along with the anchors. No clarification of decrease given; in any case, that prerequisite is to alter the concrete to represent the bending of anchor inside the grouting pad and the impediments on grout pad thicknesses not given. AISC joined checks for shear and bending are made on the anchors, and subsequent area of the anchors is 20% bigger than those rods lacking shear.

The shear capacity computed as per the concrete breakout for a typical group of cast-in anchors formulated as

$\varphi V_{cbg} = \varphi(A_v/A_{vo})\psi_5\ \psi_6\ \psi_7 V_b$

where $\varphi = 0.70$

$V_b = 7(l/d_o)^{0.2}\sqrt{d_o}\sqrt{f_c'}c_1^{1.5}$

c_1 = edge distance accounted in load direction

f_c' = compressive strength of concrete

l = the depth of embedment

d_o = the diameter of the anchor rod

ψ_5 = 1 when all anchor are to have the same load

ψ_7 = 1.4 when the concrete is intended to be with proper supplementary reinforcements or uncracked

The value recommended for the diameter of the anchor rod (do) in the formula of Vb to be limited 1.25 in. The governing factor would be shear strength of the anchor rod only if edge distance (c1) is sufficiently vast enough. If threads excluded from the plane of shear for a single anchor rod's nominal shear strength is computed equal to 0.4FuAr, and if there is an inclusion of the threads then the nominal shear is 0.5 FuAr. ACI 318 perceives the benefits of friction and permits allotment of the anchor's shear with the friction created from the flexural and axial factored load.

Checking for a breakout of the anchor near to the edge or that anchor which are deeply embedded should be done for the evaluation of the concrete breakout strength. The load consideration for a breakout for both inner and outer anchors to be equal and if breakout of concrete is restricted to the outer two anchors then all outer anchors are to be considered to take all of the shear. To achieve the desired ductility and shear strength, it would be necessary for the breakout cone anchored by the use of reinforcements. The pry out strength hasn't found in any common cases where it controls a typical design of anchors. Therefore as per ACI the pry out strength is defined as

$V_{cp} = k_{cp}N_{cp}$

Where $k_{cp} = 1.0$ $\{h_{ef} \leq 2.5\text{in.}\}$

$k_{cp} = 2.0$ $\{h_{ef} > 2.5\text{in.}\}$

N_{cp} = single anchor's nominal concrete breakout strength

h_{ef} = effective embedment length of anchor

4.3.1.3 BS 5950-1:

The anchor/holding down bolts is provided in a steel base plate to counter the effects from those factored loads discussed in the earlier chapter. Since there is a moment applied to steel base plate by column section tension occurred in bolts is to be checked as well as design should be supporting uplift caused due to applied forces.

A washer plate should be provided for the anchor bolts to repel the tension or introducing embedment of other load distributing member into the foundation. The length of the embedment of the anchors and the distributing load assembly provided should be in a way that the forces of the anchorage transfer should not exceed the foundation's load capacity. The span of this element should be efficient enough to support those adjustment tubes or the grout tube, may provide in the assembly. Using Pt which is the tension capacity is given by

$P_t = 0.8p_tA_t$

wherein,

p_t = the bolt tension strength which is given in the below table

A_t = area of tensile stress as per the specs of standard bolt. If it's not listed then the value should be taken as the area of the thread bottom.

Bolt grade		**Tension strength p_t (N/mm²)**
4.6 8.8 10.9		240 560 700
General grade HSFG to BS 4395-1	≤M24 ≥M27	590 515
Higher grade HSFG to BS 4395-2		700
Other grades ($U_b \leq 1000$ N/mm²)		$0.7U_b$ but $\leq Y_b$
NOTE 1 U_b is the specified minimum tensile strength of the bolt. NOTE 2 Y_b is the specified minimum yield strength of the bolt.		

TABLE 4.1: Bolt Tension Strength (BS5950-1)

The bolts provided should be competent enough to transmit the horizontal shear forces that occurred between the foundation and the respective column. This statement can only be agreed if the process can happen through one of the following:

- The resistance from friction occurred between the foundation and steel base plate

- The provided bolts should have enough shear resistance to allow the resistance caused by the concrete.
- Shear resistance of that surplus part of the foundation around the steel base plate.
- Block or bar shear connectors which are some of the elements for resisting shear forces.

In case if an expanding or the resin grouted type anchor bolts provided, the assembly should be capable of carrying the load applied on them. It should be noted that rag bolts or the foundation bolts should not used for tension resistance. If the contact area of the column base end and the baseplate are in tight contact bearing the transmission of the compression may be done through direct bearing on the base plate. For that tension or the shear developed in the connection because of the combinations of the factored loads weld or bolts are provided for transmission.

4.3.2 Moment:

4.3.2.1 EC-3:

The weld is designed by typically assuming that the bending moments carried by flanges and the shear by the web. The welded design for the web of the section should be equipped to carry base shear. There is no particular direction it may act in both directions that is there is no compression flange. If the case considered for compression, then the column member's saw end is typically sufficient for the contact in direct bearing hence nominal weld s of 6mm or 8mm are generally required. In the tension flange design force considered is the lesser value of the following

Resistance for the flange tension = $b*t_{fc}*f_y$

Force take to be moment in flange which is decreased by compression effect

$= (M_{Ed}/h_c - t_f) - N_{Ed}(bt_{fc}/A)$

4.3.2.2 AISC:

The steel column base plate with small moments with the equivalent eccentricity e, equally considering to the moment Mu, isolated by the column axial force Pu is how the design process is related. The axial force opposed by bearing as it were for small eccentricities. It's essential to utilize anchor rod for large eccentricities. The discussion of large and small eccentricities in uniform bearing stress. The strategic method have been formulated in way to be used for both ASD and LRFD. The product of qY defines the bearing force

$q = ʃ_p \times B$

where, $ʃ_p$ = The bearing stress in b/w concrete and plate

'B'is termed as baseplate width

In the area of bearing of an assembly's midpoint is where the force otherwise it's that distance which is 'Y/2' to the left .Hence the expected resultant of the distance is computed to the centerline's right of plate (ε) is

$\varepsilon = (N/2)-(Y/2)$

From this computed value it is understood that as the ε increments the Y diminishes where Y will reach its littlest esteemed value when maximum value of q achieved

$Y_{min} = P_r/q_{max}$ $[q = ʃ_{p(max)} \times B]$

The articulation, for the force of bearing which is the resultant where the location demonstrates that ε achieves it's maximum value when the term 'Y ' is min., hence

$\varepsilon = (N/2)-(Y_{min}/2) = (N/2)-(P_U/2q_{max})$

The line of activity of the applied load, Pu, for moment equilibrium and of force by bearing, 'q Y' must concur; which terms as,

$e = \varepsilon$.

Surpassing of eccentricity ($e = M_r/P_r$) would be the maximum significance that ε can achieve, loads applied can't be opposed alone by bearing and tension is the state in which the anchors will be. In synopsis, for estimations of e not exactly ε_{max}, Y is more prominent than Y_{min} and so q_{max} will be more than that of q hence the $ʃ_{p(max)}$ will be greater than $ʃ_p$. The value for e is more than ε_{max} then $q = q_{max}$. In this way, the load combination of an eccentricity critical value is

$$e_{crit} = \varepsilon_{max} = (N/2) – (P_r/2q_{max})$$

While breaking down plate configurations and different load, on the off chance that $e \leq e_{crit}$, there will be no tendency for any overturning, hence anchors aren’t essential for equilibrium of the caused moment, and forces that are in combination will be considered to have a little moment. Then again, if $e > e_{crit}$, moment equilibrium can't be kept up alone through bearing then anchors are necessary. Combinations of such moment and load in axial are alluded to as cases of the moment which is large.

Interface of Bearing for Base Plate Flexural Yielding Limit

At the cantilever length base plate bending is caused duo to the bearing pressure in between the base plate and concrete. The notations for strong axis bending and weak axis bending is taken to be m & n respectively. Thus the bearing tension for the strong axis is computed as

$$ʃ_p = (P_r/BY) = P_r/B(N-2e)$$

The base plate strength required is

$$M_{pl} = ʃ_p(m^2/2) \quad \text{(for the case when } Y \geq m\text{)}$$

$$M_{pl} = ʃ_{p(max)}\, Y(m- (Y/2)) \quad \text{(for the case when } Y<m\text{)}$$

M_{pl} = bending of plate per unit width

The plate's nominal resistance is

$R_n = F_y t_p^2/4$

F_y = The given yield stress of the material of the plate

t_p = the thickness of the plate

The plate's strength available is calculated as

$(R_n/\Omega) = (F_y/\Omega)(t_p^2/4)$ (for ASD)

$\Phi_b R_n = \varphi_b F_y(t_p^2/4)$ (for LRFD)

Φ_b = 0.90 = strength reduction factor (bending)

Ω = 1.67 = safety factor (bending)

To calculate the thickness of the plate the following the procedure is done with the use above formulated equations

Case 1: $Y \geq m$:- $t_{p(req)} = \sqrt{(4\{ʃ_p(m^2/2)\}/F_y/1.67)} = 1.83m\sqrt{(ʃ_p/F_y)}$ (for ASD)

$t_{p(req)} = \sqrt{(4\{ʃ_p(m^2/2)\}/0.90F_y)} = 1.5m\sqrt{(ʃ_p/F_y)}$ (for LRFD)

Case 2: $Y<m$: - $t_{p(req)} = 2.58\sqrt{(\{ʃ_p Y(m- (Y/2))\}/F_y)}$(for ASD)

$t_{p(req)} = 2.11\sqrt{(\{ʃ_p Y(m- (Y/2))\}/F_y)}$(for LRFD)

The thickness would be represented by n when it seems to be bigger in value than m. While this methodology offers a straightforward method for design planning the baseplate as for bending when plate thicknesses governed by n, it is suggested to use different strategies for the plate for flexure designing.

Tension for Base Plate Flexural Yielding

In the anchor rods there will be no tension if the moment such that $e \leq e_{crit}$ and thus in tension interface it will not cause the base plate to bend hence the governing factor of the baseplate is the bearing.

Now those baseplates under large moments the anchors are connected to the' baseplate to that of the foundations when the bending moment has a magnitude larger than the axial load through column so as to prevent the base from failing nor tipping over during the bearing the concrete at the compressed edge. Large moments expected even if the rigid frames are designed to withstand the wind or earthquake loads if

$e > e_{crit} = (N/2) – (P_r/2q_{max})$

Interface of Bearing for Base Plate Yielding Limit

$ʄ_P = ʄ_{p(max)}$ (bearing stress is at a definite value for moments which are considered large)

The following are the formulae to compute the value for the thickness of the base plate for both ASD and LRFD cases. Note that if n is to be larger than m, the thickness would govern and computed with the formulae by the larger value of n and m.

Case 1: Y≥m

$t_{p(req.)} = 1.83m\sqrt{(ʄ_{p(max)}/F_y)}$ for the case ASD

$t_{p(req.)} = 1.5m\sqrt{(ʄ_{p(max)}/F_y)}$ for the case LRFD

Case 2: Y<m

$t_{p(req)} = 2.5\ 8*(\sqrt{(ʄ_{p(max)}Y(m-Y/2)/F_y)})$ for the case ASD

$t_{p(req)} = 2.1\ 1*(\sqrt{(ʄ_{p(max)}Y(m-Y/2)/F_y)})$ for the case LRFD

Interface of Tension for Base Plate Yielding Limit

The bending of the base plate effects or caused by the tension force. Cantilever activity minimally expected with that of the range length equivalent to that of separation from rods middle line to that of the flange's focal point of a column, on other hand assumption of the bending lines can be proceeded. The computation of the bending strength required for the unit width of a plate formulated as

$x = ʄ - (d/2) + (t_ʄ/2)$

$M_{pl} = T_a x/B$ (for case of ASD)

$M_{pl} = T_u x/B$ (for case of LRFD)

d=column flange width

$t_ʄ$ = thickness of column flange

After computing the applied moment is equal to the strength the provided formula to find the thickness of the baseplate is as follows

$t_p = 2.58\sqrt{}T_a x/BF_Y$ (for ASD)

$t_p = 2.11\sqrt{}T_u x/BF_Y$ (for LRFD)

4.3.2.3 BS 5950-1:

A uniform pressure should be assumed while calculating for the applied moment force on the steel base plate by the column where the uniform pressure shouldn't exceed the value of the strength of bearing for concrete and the $p_{yp}S_p$, where the plastic modulus of the base plate is the S_p. If a stiffener provided in the steel column base plate, then the moment caused by the bearing pressure shouldn't exceed the value of $p_{ys}Z_s$ wherein the stiffener design strength is the p_{ys}. If the gross area is more than that of the effective area of the base plate, then the connection needs to be checked for the effects of the linear distribution of the pressure on the base plate, stiffeners and the gross area as well.

If the connection is not having a suitable contact surface for the transmission of the compression through direct bearing, then the bolts or welds should be provided for the transmission of all moments and the forces applied in the connection.

4.3.3 Axial:

4.3.3.1 EC-3:

The value chosen for the compression would be the value which would be lesser among the following resistance which are

- The bearing resistance of the foundation
- The bending resistance of the base plate

The first condition depends on the effective area where the dispersion of the forces due to compression ensuing from the bending of the baseplate. Thus distribution of force is restricted by bending resistance of plate as well as dimension of it. This area defined as the additional bearing width around the steel section's perimeter.

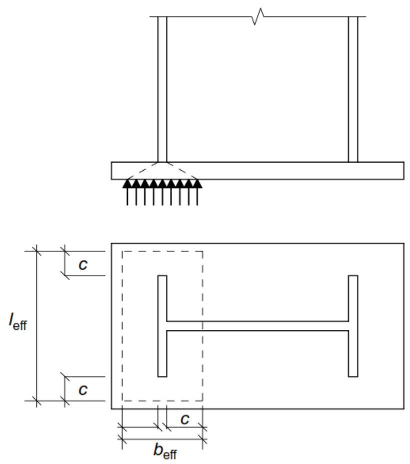


FIGURE 4.6: Effective Bearing Area (EC-3)

Thus the design for a resistance by compression of foundation is formulated as

$F_{C,Rd} = f_{jd} b_{eff} l_{eff}$

Where

$f_{jd} = \beta_j \alpha f_{cd}$=   the joint's design bearing strength

$\alpha = \min[(1 + (d_f/\max(h_p,b_p);(1+2(e_h/h_p));(1+2(e_b/b_p));3]$

d_f = concrete foundation depth

$f_{cd} = \alpha_{cc}(f_{ck}/\Upsilon_c)$

α_{cc} = 0.85 as per B S-E N-1992-1-1[10]

Υ_c = concrete material factor essentialy it is equal to 1.5 as per UK NA

B_{eff} and and $/_{eff}$ are shown in the above figure for reference

c = is the base plate width limit

β_j = 2/3

As per B S-E N-1993-1-8 for using the value of β_j as 2/3 it is required to have :

- The compressive strength of the grout should be at the very minimum of 0.2 f_{cd} and atleast equalvelent to f_{cd} if more than 50mm thick.
- The grout thickness needs to be less than $0.2h_p$ and $0.2h_p$

If the foundaton dimensions are not known then $\alpha = 1.5$ can be assumed hence

$f_{jd} = f_{cd} = 0.85(f_{ck}/\Upsilon_c)$

Bending resistance of base plate also stated as resistance' of baseplate which limits added width of 'c,' where the width 'c' is assumed to be a cantilever that imperiled to uniform load which is the same as the bearing strength designed for that particular joint. The factors like yield strength and thickness which affect the bending resistance of the column base plate in such the limiting addition width formulated as

$c=t(f_y/3f_{jd}\lambda_{MO})^{0.5}$

The compression resistance and that web in the zone of compression is computed as:

$F_{c,fc,Rd} = M_{c,Rd}/h_c\text{-}t_{fc}$

The other regulations are only applicable for a single row bolt which are found to be outside of the tension flange. Suppose extra bolts are additional provided b/w the flanges then these can be treated similar to that of an end plate connection. The value to be taken should be the smallest possible from the following condition for that of resistance' of tension in a T-stub

- Baseplate's resistance when it's in that of bending
- Holding-down bolts resistance
- Web and the flanges of a column's resistance when it's in a state of tension

Bending State of Resistance of Base Plate

The procedure for a bending resistance is determined as

$F_{t,pl,Rd} = 2 * M_{pl,1,Rd} / m_x$

where $M_{pl,1,Rd} = 0.2\,5 * \sum l_{eff} t_{pl}^2 f_y / \Upsilon_{MO}$

l_{eff} = an equivalent of T-stub's effective length

t_{pl} = baseplate thickness

f_y = baseplate yield strength

m_x = That distance b/w bolt centerline and the fillet weld to the flange of the column

The yield patter are adviced to be checked if the corner location in which bolts are positioned at the tip outisde of that flanges of column and try to match with the following table and check whther its still appropriate.

Tension State of Resistance of bolts

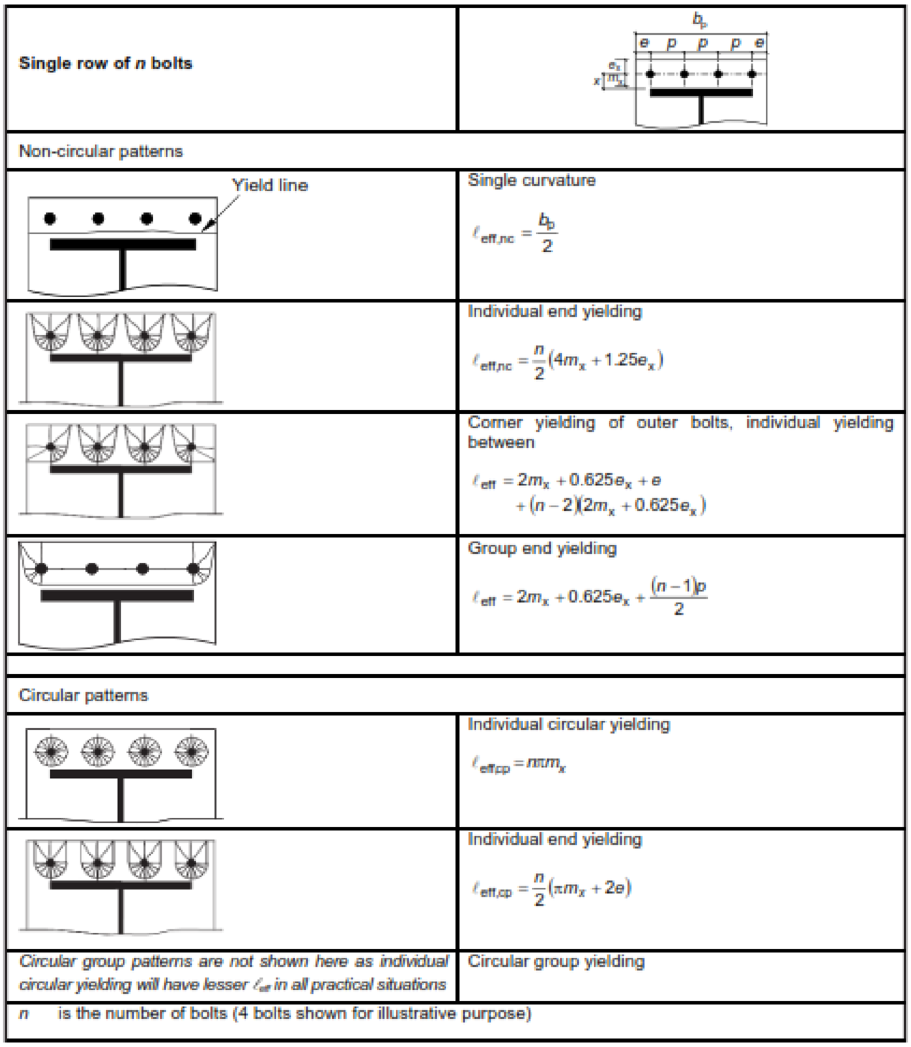
For a single bolt rows the equation thus formulated is as follows

$F_{t,pl,Rd} = nF_{t,Rd}$

Where, in n = bolt numbers

$F_{t,Rd}$ = a single bolts tensile resistance design

Suppose there is a second bolt row that positioned inside the tension flange, so a triangular distribution way of approach should be done to limit the resistance of these bolts from the rotation center as for the bolts which are in an endplate connection the outer tension row resistance is checked by a Mode 3 failure



Single row of *n* bolts	
Non-circular patterns	
Yield line	Single curvature $\ell_{eff,nc} = \frac{b_p}{2}$
	Individual end yielding $\ell_{eff,nc} = \frac{n}{2}(4m_x + 1.25e_x)$
	Corner yielding of outer bolts, individual yielding between $\ell_{eff} = 2m_x + 0.625e_x + e + (n-2)(2m_x + 0.625e_x)$
	Group end yielding $\ell_{eff} = 2m_x + 0.625e_x + \frac{(n-1)p}{2}$
Circular patterns	
	Individual circular yielding $\ell_{eff,cp} = n\pi m_x$
	Individual end yielding $\ell_{eff,cp} = \frac{n}{2}(\pi m_x + 2e)$
Circular group patterns are not shown here as individual circular yielding will have lesser ℓ_{eff} in all practical situations	Circular group yielding
n is the number of bolts (4 bolts shown for illustrative purpose)	

TABLE 4.2: Effective Length for Base Plate (EC-3)

4.3.3.2 AISC:

This section carries out the design of the anchor rods for the tension. From the point of fundamental analysis of the working for endorsed loads over a building, the net maximum uplift for the columns obtained. At the point during an uplift caused due to wind surpasses over dead-weight of a rooftop, the supporting' sections are exposed to overall forces of uplift.

Moreover, due to overturning the column segments in braced bays or rigid bents might be exposed to net uplift force. Hence the determination of the maximum uplift in for a column due to such forces is critical.

The anchor's quantity intended is an element of the most extreme overall uplift on that of the column section and the strength quality per anchor for its rod material picked. The material chose for the rods should conform to the material discussed as per standards in the previous chapter. Ordinarily, the prying force effects in the anchor rod ignored. Which is usually justified when' the baseplate thickness is determined accepting bending cause of cantilever around' web and additionally flange of that of the section (column), and because the length' of anchor rods result in bigger deflections' than for connection between the steel. So by a selection of the material, quantity, and size of anchors for the uplift to be resisted plays a vital role in the design of the anchor rods as stated.

With tensile loads or compressive forces, the bending caused governs the base plate thickness. A basic methodology for the effect of the tensile loads is by assuming the anchor rods loads produce bending moment in the baseplate steady with action cause of cantilever about the flange or the web one-way bending of the column. In an event where a web of a section is encountered anchors from baseplate, web and its connection to baseplate ought to be confirmed. Then again, a better baseplate investigation for anchors situated inside of that of the segment flanges of the intended section can be utilized to consider about both the web' and' the' column' section flanges bending in both the ways. For those of the two-way type bending methodology, the inferred bending moment ought to be steady with similarity necessities for the base plate deformations. So in any case, for the baseplate effective bending be unobtrusively estimated utilizing a 45° conveyance from that of the middle line of the anchors to face substance of the column segment flange/web. This approach aids in the thickness, size, and welding of the base plate to transferal from the uplift forces caused.

An anchor rod has its tensile strength equivalent to the concrete anchorage strength of the anchor' rod cluster, or it's those anchors that are active in tension when there is a moment created otherwise it can directly put as the equivalent of the total steel tensile strength of the participating anchors. Hence those rods in tension the tensile strength which have been designed for the anchor rods to contribute is engaged as the minimum of those total of steel tensile strength of a participating anchor rod individually else the strength of that tensile of the concrete in the anchor set. As per current American Concrete Institute criteria, the concrete tensile strength or length of distorted rods computed.

The dependence of the tension which is limiting on the anchors on its area-based which is minimum along the length which is the largest stressed of that particular rod, this is ordinarily inside the threaded part with the exception of when upset rods are utilized.

Tensile stress area = $(D – 0.7854/n)^2$

As per the code ANSI/ASME defines this threaded area where D = main dia. & n = no. of threads in inches.

For deciding the required tensile stress region, two techniques commonly utilized. Firstly, the dependence on a specially formulated code like The American Society of Mechanical Engineers - A.S.M.E which are stipulated' area of tensile' stress as depicted above. The other is to include a factor of modification that would relate the area of tensile stress straight to the unthreaded' region as a method of streamlining the procedure for design. Strength of that of the structural' fasteners has truly been founded over the bolt distance across, and the direct approach of stress cause of tensile is stipulated. The designer ought to know about the distinctions in configuration methodologies and remain predictable inside one framework while deciding the required anchor area. Be that as it may, the determined strength of a specific anchor examined by either technique would deliver a reliable final product. Both designing methods have strength formulated tabulation for usually utilized rod (anchor) materials as well as sizes that are effectively created by the strategies would pursue.

The nominal tensile strength of a fastener is determined as per the AISC (2005 specification)

$R_n = 0.7F_uA_b$

For LRFD it's computed with the use of φ=0.75

Hence the design tensile strength can be calculated as

$= 0.75^2F_uA_b = 0.5625F_uA_b$

For ASD it's computed with the use of Ω=2

Therefore Allowable tensile strength = $(0.75/2.00)F_uA_b = 0.375F_uA_b$

As per ACI the design tensile strength is computed as = φF_uA_{ts}=$0.75F_uA_{ts}$

Where φ taken as 0.75

At the point when an end of a column base opposes just compressive loads caused through axial over a column, the baseplate should be sufficiently vast to oppose the bearing forces exchanged from the baseplate which is limit of bearing of the concrete, and sufficient thickness for the baseplate expected that is base plate yielding limit.

FOR W-SHAPES-BASEPLATE YIELDING LIMIT

For those baseplates which are loaded axially, the tension due to bearing beneath the baseplate is expected consistently appropriated and written as

$f_{pa} = P_a/BN$ as for ASD

$f_{pu} = P_u/BN$ as for LRFD

Likewise, this pressure of bearing leads to bending caused in the baseplate in the zone/area among the flanges of a column. The following procedure enables a single strategy to decide the baseplate thick for the two circumstances. The baseplate required strength calculated through the preceding equations

$M_{pl} = f_{pa}(l^2/2)$ as per ASD

$M_{pl} = f_{pu}(l^2/2)$ as per LRFD

the baseplate critical cantilever side dimension 'l' is largest of λn', n, and m

$m=(N-(0.95*d))/2$ & $n= (B-0.8b_f)/2$

$\lambda n' = \lambda(\sqrt{d b_f})/4$

λ can be taken as 1 which seems to be conservative

N = length of the base plate in inches

B = width of the base plate in inches

b_f = width of the column flange in inches

d = column overall depth in inches

n' = Theory of the yield-line distance of cantilever which is from the column flange or web (in)

$\lambda = (2\sqrt{X})/ (1 + \sqrt{(1-X)}) \leq 1$

$X = [(4db_f)/(d+b_f)^2]*(\Omega_C P_a)/\varphi_c P_p$ as for ASD

$X = [(4db_f)/(d+b_f)^2]*(P_u)/\varphi_c P_p$ as for LRFD

Where, P_a = the axial compressive load which is required by ASD

P_u = the axial compressive load which is required by LRFD

$P_p = 0.85ʃ_C'A_1(\sqrt{}A_2/A_1)$

The minimum base plate thickness for the limiting yield can be equated as

$t_{min} = l\sqrt{}(2\Omega P_a/\varphi F_yBN)$ case for ASD

$t_{min} = l\sqrt{}(2P_u/\varphi F_yBN)$ case for LRFD

where, $\varphi = 0.9$ flexure resistance factor

$\Omega = 1.67$ F.O.S for ASD

F_y = base plate minimum yield stress as specified (ksi)

From the values of m, n, and λn ',' l' has the maximum value hence for the base plate the thinnest value can initiate by limiting m, n, as well as λ. This is normally attained by adjusting the baseplate measurements with the goal that m and n approximately equivalent.

BASE PLATE YIELDING LIMIT (HSS AND PIPE)

To calculate the m and n of a rectangular hollow section the depth and the width of it is buffed by 0.95 times of yield lines. The alteration for n and m should be made for calculation of a HSS (Hollow Structural Sections) column. As for the round hollow section the determination of n and m are by utilizing yield lines at 0.8 times the dia. in which case the term λ isn't used for the designing.

4.3.3.3 BS 5950-1:

The thickness should not be less than the value t_p when a concentrically applied axial forces to the base plate where in the

$t_p = c[3w/p_{yp}]^{0.5}$

wherein,

p_{yp} = the baseplate design strength

w = the pressure produced under the steel base plate as per the assumption of the uniform dispersal of pressure all over the portion which are effective.

c = is that perpendicular distance which is biggest between the face of column and the portion which is effective of the steel baseplate.

t_p = Maximum or usually the flange thickness of the section as in the column.

If the steel base plate is eccentric to the column section then the applied axial force produce the moment which shouldn't surpass the value of $p_{yp}Z_p$, the base plate section modulus is Z_p.

4.4 Inference from the Parametric Study:

The parametric study carried out in the section was for the significant steel building design codes followed in the world which are the Eurocode (EC-3), American Institute of Steel Construction (AISC) and the British Standard Institute (BS 5950-1). Chapter 3 and chapter 4 of this work have shown that each code is critically important in different aspects of the design. As discussed in the above chapters, a general comparison and the parametric differences were discussed to make the agenda of switching codes easier for the structural engineers easier. It is preferred to follow the units prescribed in each code that need to attend to apart from the design philosophies and the empirical formulae that have been inscribed. Note that more checks are available through the AISC design code when compared to the other principal international codes. This approach is to have a safer method of design. Sample calculations attached in APPENDIX B, C, and D for BS 5950-1, AISC and EC-3 respectively of the codes for corollary. From the calculations, infer that Eurocode has a comparatively smaller base plate which satisfactory. However, the AISC seems to be more optimum as the thickness of it is 12mm and is lesser than the B.S. code design, which is 20mm even though the length and the widthwise B.S. code is superior. Note that the thickness of any plate plays an essential role in the fabrication point of view as the weight of it is mostly depended on it hence AISC design code seems to have the upper hand than the B.S. code.

In the country of the United Arab Emirates, the codes which have predominantly followed are the British standard and then the American standard steel building design codes. These are the two codes that have been approved through the local authorities so far even though the Eurocode has used extensively in all over Europe. The British code have been phased out from the U.K. itself, but it is because of the vast number of engineers in U.A.E. are still using and the consultant dependence the B.S. is still held in the country. Note that slowly the American codes are demanded in a few consultants and the authorities nowadays. Hence the comparison and the review shall be limited to British and the American after this chapter.

CHAPTER 5: DESIGN ANALYSIS OF A STRUCTURAL STEEL WAREHOUSE USING BS5950 AND AISC

5.1 General:

In this chapter, a critical analysis of the two chosen international steel building design codes BS5950 and AISC used for the computer analysis approach of a steel warehouse. The content in this section will include details of building and evidence through software outputs to support findings and comparisons made through earlier chapters. As the previous chapter's theory based approach along with the empirical formulae used. So the results from the computer analysis are cross-checked and concluded how far the theory approach is in harmony with the actual design. There are two types of base plate used for a column base which are a pinned and the fixed connection depending upon the region. As far as in U.A.E, the region is not susceptible to any seismic effect the most common base connection provided is the pinned base plate connection hence the focus shall be kept to only the pinned type steel column base plate.

Boundary conditions in the structural analysis are real. A column base typically idealizes the model with beams or truss elements as either fixed or pinned boundary conditions. The occurrence of an error in the computer drifts expected if there is an improper characterization which would lead to a 2nd order moments which are unrecognized only if those of the misjudged stiffness, or too much column size if the miscalculated stiffness. To get more precise and accurate design results, then the plastic and elastic stiffness inputs of the column baseplate connection are mandatory. The type of the column required baseplate connection detail chosen affects the structural analysis forces and deformations, which are used to design the connection for the column base plate.

The computer analysis so forth done is a representation of which code would be more relevant in terms of economy and comparison is done concerning the parameters discussed in the previous chapters. This chapter is an attempt to relate the theory which has put forth and the computer analysis output.

5.1.1 Design Philosophy:

The purpose of this design calculation is to design the structural steel for steel warehouse for its integrity, strength, and stability verification. The result of this conclusion is that the structure considered adequate in meeting the required design criteria.

5.1.2 Unit of Measurement:

The unit of measurement in design shall be in the Metric system for the convenience of all the codes.

5.1.3 Design Code and reference:

Dead load and imposed load calculated as per BS 6399-1 & AISC for the structural loadings.

5.1.4 Computer Analysis Software:

- STAAD-Pro V8i for Structural calculations
- Master Series for steel column base plate calculation for BS 5950
- RAM Elements for steel column base plate calculation for AISC

5.2 Model:

The model created in for the analysis is a steel warehouse with a 30 slope with an A-Frame. The building also consists of the first floor added. The sections used in the building are the British sections. Which comprises of both hot rolled and cold rolled sections. The steel grade chosen is S275JR. The column base is considered to be pinned base connections.

The dimensional aspect of the structure is 36m in width and 30m in length; the eve height of the building is 7.70m. The bay spacing of the warehouse is 6m in space. The rood is 30 in slope as mentioned. The flooring of the first floor laid with a metal deck. The section profiles used in the structure is as follows:

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE.

MAIN STRUCTURE

For columns = UB 762x267x134

For rafters = UB 762x267x134

For bracings ties, and struts = 168mm outside diameter with 8mm thickness pipe (6"inch pipe)

FIRST FLOOR OF THE STRUCTURE

For the main frame of the floor = UB 610x229x113

For the secondary members = UB 406x178x54

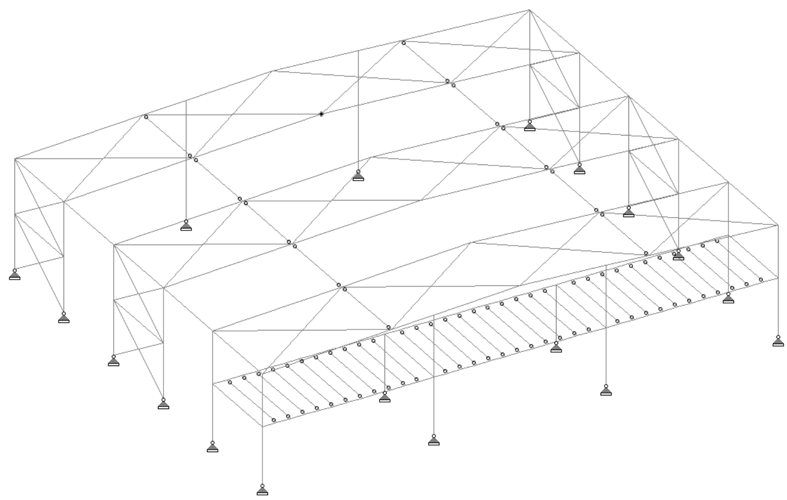


FIGURE 5.1: Wireframe View of the Steel Structure

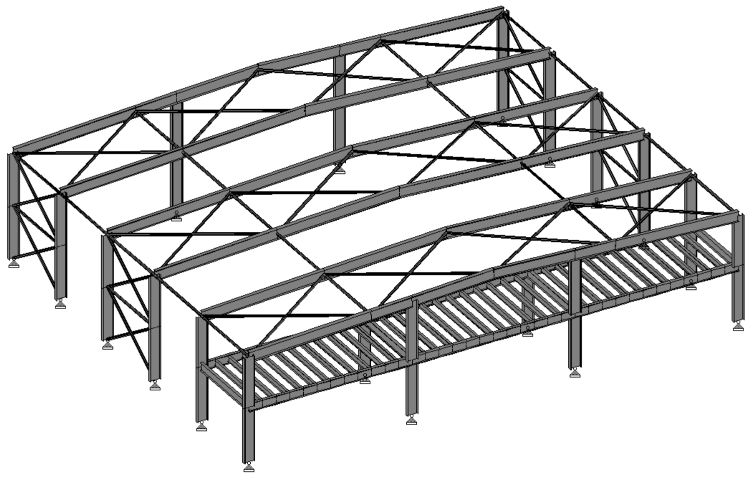


FIGURE 5.2: Rendered View of the Steel Structure

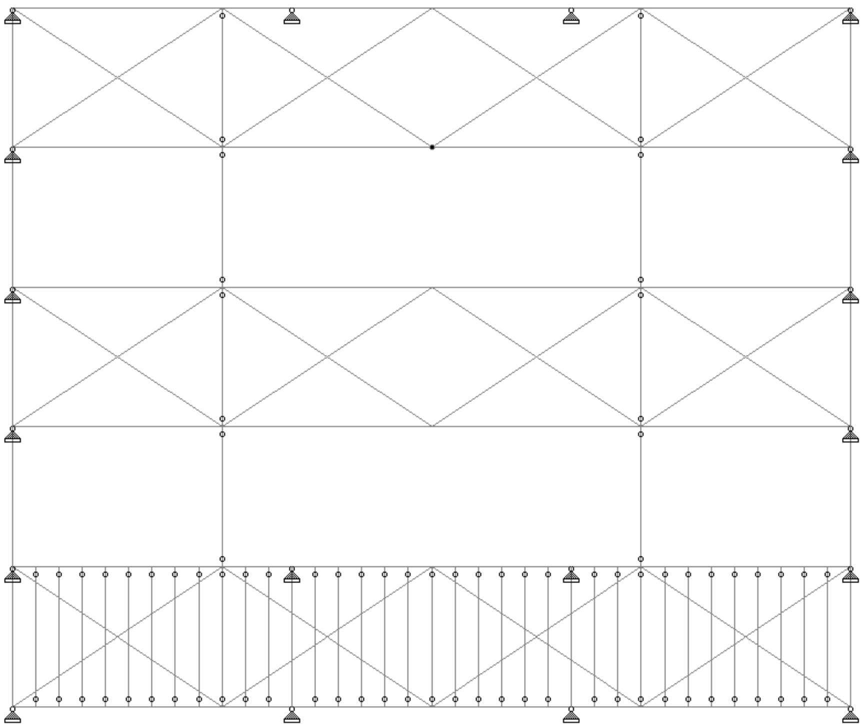


FIGURE 5.3: Plan View of the Steel Structure

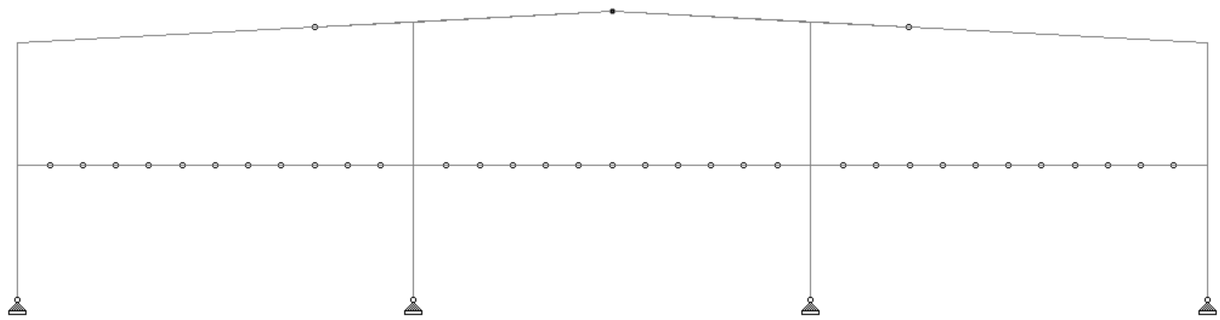


FIGURE 5.4: Elevation View of the Steel Structure

5.3 Loadings of the building:

The structure involves two principal loadings, which are the dead loads and the imposed loads. However, the dead loads identified as the dead load for the self-weight of the structure and super dead load for the non-structural elements. The imposed load denoted as the live load in the software analysis.

Dead load (DL):

The structural dead load of the members involved in the building is taken self-weight multiplier default as 1 by the software concerning the code chosen for the analysis.

Super Dead load (SDL):

As discussed the super dead load involves the dead weight of the non-structural elements which are:

- Cladding and Fixation = 0.15kN/m^2
- Metal deck for the first floor = 10 kg/m^2

Imposed (Live) load (LL):

The live load taken are by the two codes for the first floor

As per BS 5950-1 = 4kN/m^2

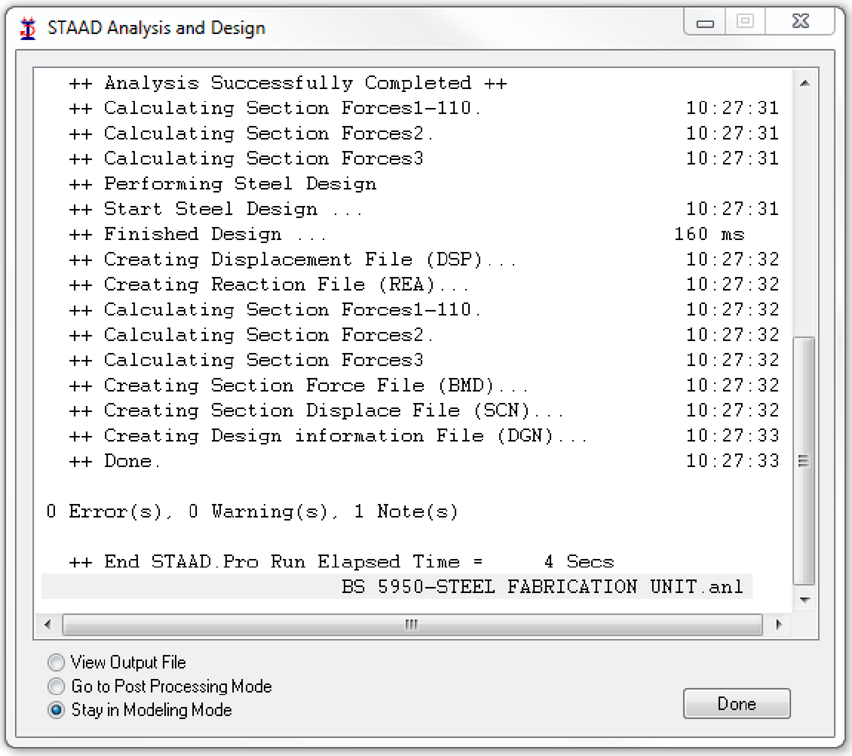
As per AISC = 3.59kN/m^2

5.4 Load Combinations of the building:

The factored load combinations is however discussed in the earlier chapter an shall be used in the software itself as per both codes BS 5950 and AISC

AISC	*BS 5950-1*
1.4DL	1.4DL+1.6LL
1.2DL+1.6LL	1.4DL+1.4WL
1.2DL+LL	1.2DL+1.2LL+1.2WL
1.2DL	
0.9DL	

TABLE 5.1: Load Combinations Used for Computer Analysis



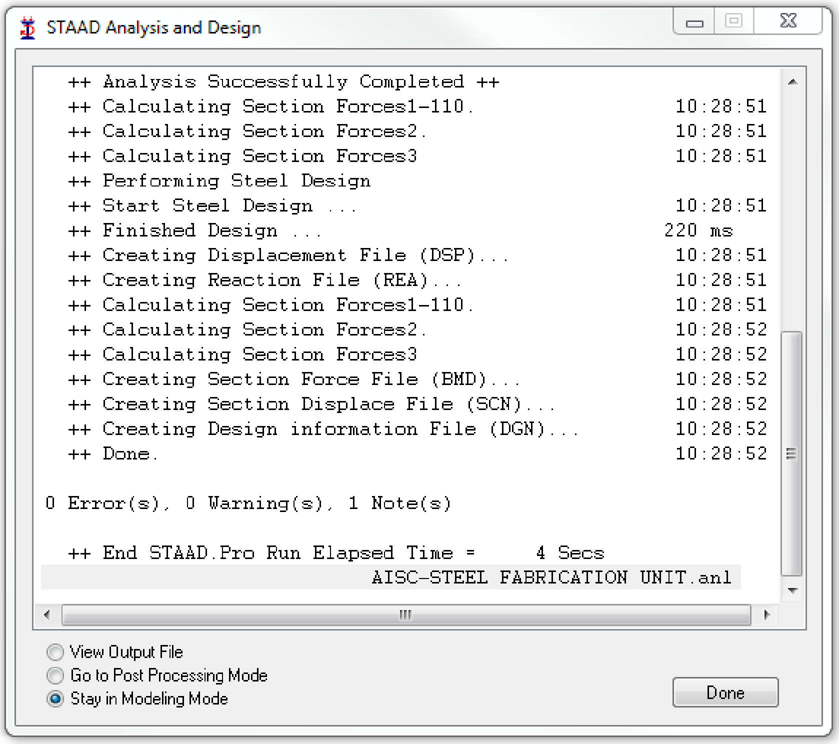


FIGURE 5.5: Model Error checks for both BS 5950 and AISC respectively

5.5 Model Analysis:

5.5.1 Member Checks:

The same section sizes used for both the BS and the AISC analysis. However, AISC analysis failed in analysis, and members had to change to make structure safe, which in turn increased the weight of the structure.

From figure 5.5, there were no model errors encountered, so the analysis has done accurately.

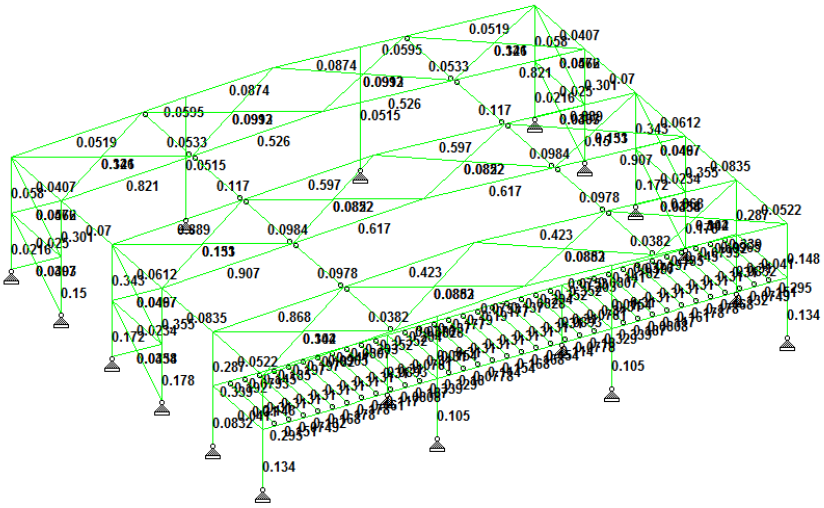


FIGURE 5.6: BS-5950 STAAD Model

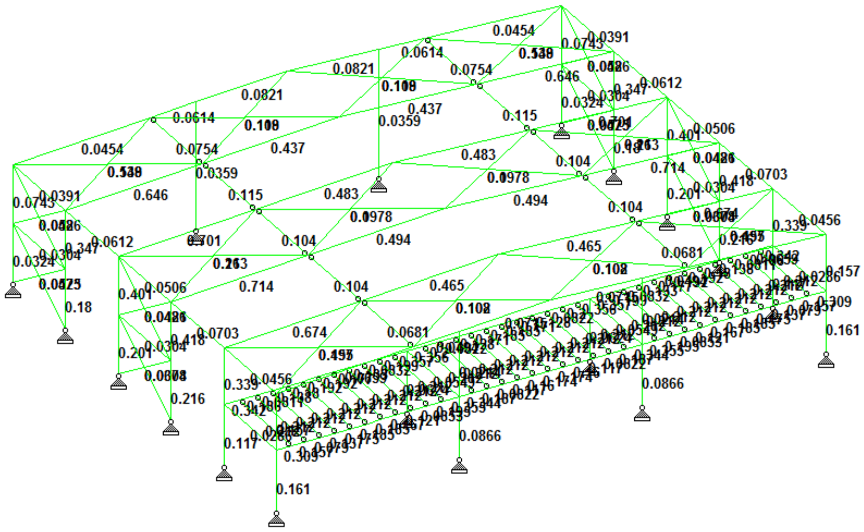


FIGURE 5.7: AISC STAAD Model

Figure 5.6 is the BS version of the model analysis and the initial chosen members sizes passed in the given loads and the conditions with a ratio of 0.9. However, the AISC model has a ratio of 0.714, which is much lesser than the BS design, and the weight of the structure can decrease further. From the figures, it is evident that most of the load-carrying members are the rafters, which is evident that critical loads that have applied are on those elements of the structure.

Since the initial members used for the AISC can be reduced the following members were used to make the structure safe.

MAIN STRUCTURE

For columns = UB 686x254x125

For rafters = UB 686x254x125

For bracings ties, and struts = 6inch pipe

FIRST FLOOR OF THE STRUCTURE

For the main frame of the floor = UB 610x229x113

For the secondary members = UB 406x178x54

Note that only the main column and rafters where changed keeping the rest the same this is because it was the main structural element and the utilization seems to be affecting the area of rafters. The following figure 5.8 is the weight reduced analysis of the structure.

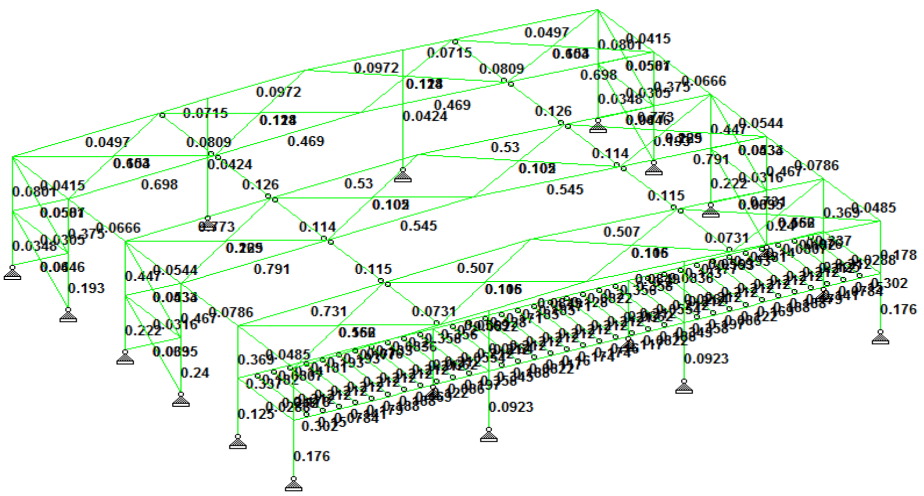


FIGURE 5.8: AISC STAAD Model (Optimum)

As per the computer analysis, the steel weight of the BS and the AISC models also vary differently since the members got changed in AISC to be safer and have a weight reduction as well. The following are the weights of both the models:

BS 5950 model = 84 Tons (approx.)

AISC model = 80 Tons (approx.)

5.5.2 Deflection Checks:

The model used for AISC will be the weight reduced model. The following table is the STAAD output of the deflection check which has been limited to L/250.

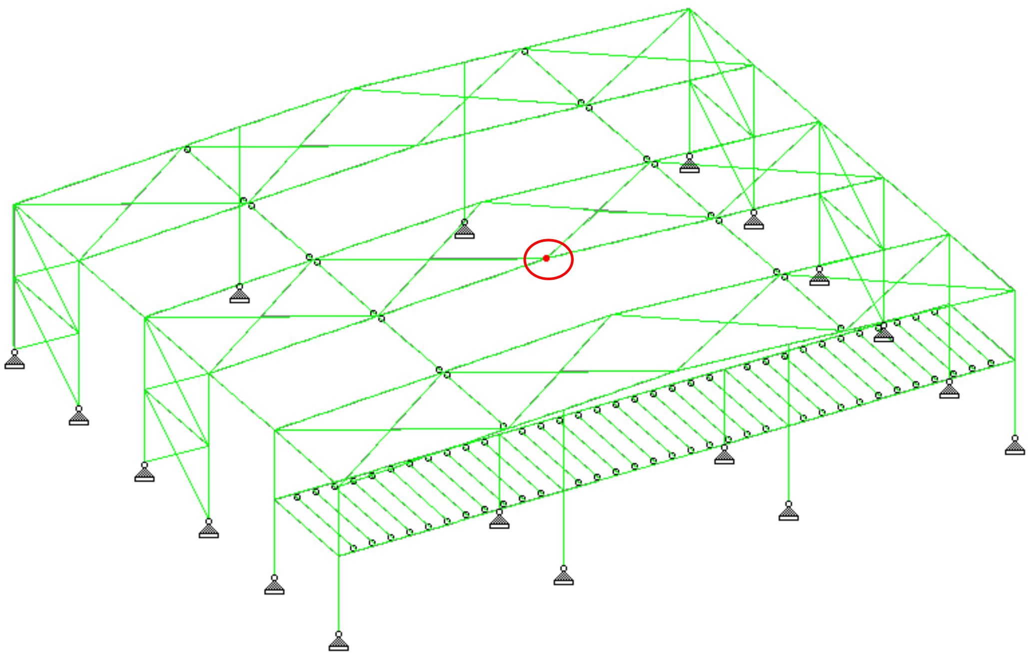


FIGURE 5.9: BS 5950 STAAD Model Deflection Check

	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	261	4:1.4DL+1.4SI	6.034	-0.116	0.140	6.036	-0.000	0.000	-0.001
Min X	260	4:1.4DL+1.4SI	-6.034	-0.116	0.140	6.036	-0.000	-0.000	0.001
Max Y	86	3:LIVE LOAD	-0.000	1.631	0.123	1.635	0.000	-0.000	-0.000
Min Y	77	4:1.4DL+1.4SI	-0.000	-73.748	1.622	73.766	-0.004	0.000	-0.000
Max Z	20	4:1.4DL+1.4SI	-0.000	-60.519	6.148	60.830	0.005	0.000	-0.000
Min Z	86	4:1.4DL+1.4SI	-0.000	-42.715	-2.942	42.817	-0.002	-0.000	-0.000
Max rX	9	4:1.4DL+1.4SI	-0.000	-1.200	5.782	5.905	0.014	-0.000	0.000
Min rX	31	4:1.4DL+1.4SI	-0.000	-1.293	-2.129	2.490	-0.012	0.000	0.000
Max rY	257	4:1.4DL+1.4SI	4.823	-0.064	-0.135	4.826	-0.000	0.001	-0.001
Min rY	256	4:1.4DL+1.4SI	-4.823	-0.064	-0.135	4.826	-0.000	-0.001	0.001
Max rZ	143	4:1.4DL+1.4SI	1.369	-46.216	0.736	46.242	-0.004	0.000	0.005
Min rZ	137	4:1.4DL+1.4SI	-1.369	-46.216	0.736	46.242	-0.004	-0.000	-0.005
Max Rst	77	4:1.4DL+1.4SI	-0.000	-73.748	1.622	73.766	-0.004	0.000	-0.000

TABLE 5.2: BS 5950 Deflection Values

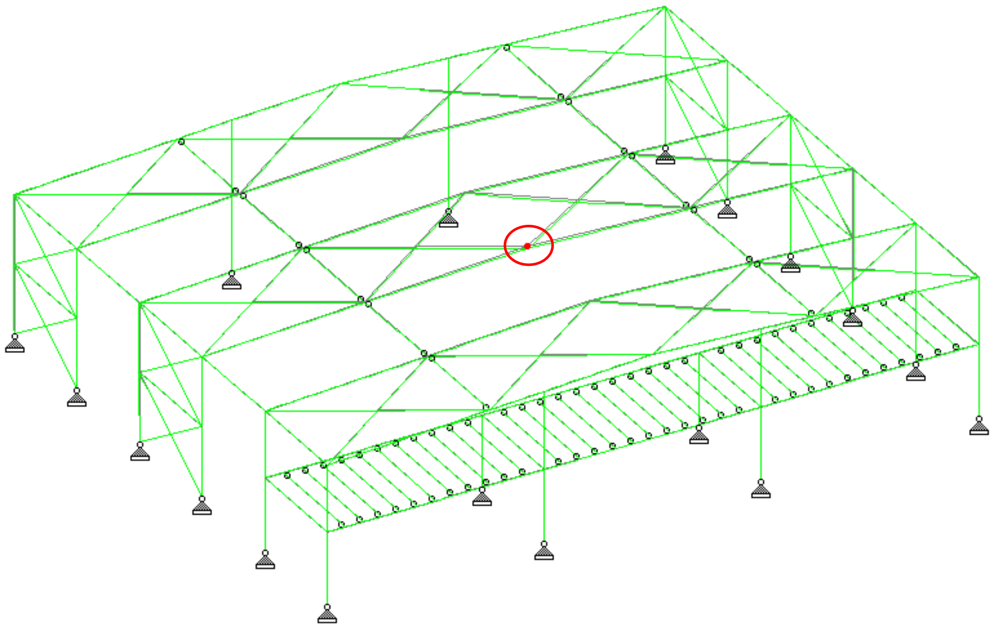


FIGURE 5.10: AISC STAAD Model Deflection Check

	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	261	4:1.4DL+1.4SI	**7.458**	-0.124	0.160	7.461	-0.000	0.000	-0.001
Min X	260	4:1.4DL+1.4SI	**-7.458**	-0.124	0.160	7.461	-0.000	-0.000	0.001
Max Y	86	3:LIVE LOAD	-0.000	**1.824**	0.149	1.830	0.000	-0.000	-0.000
Min Y	77	4:1.4DL+1.4SI	-0.000	**-90.720**	1.852	90.739	-0.004	0.000	-0.000
Max Z	20	4:1.4DL+1.4SI	-0.000	-71.319	**7.153**	71.677	0.007	0.000	-0.000
Min Z	86	4:1.4DL+1.4SI	-0.000	-52.703	**-3.586**	52.825	-0.003	-0.000	-0.000
Max rX	9	4:1.4DL+1.4SI	-0.000	-1.586	6.743	6.927	**0.016**	-0.000	0.000
Min rX	31	4:1.4DL+1.4SI	-0.000	-1.386	-2.885	3.201	**-0.013**	0.000	0.000
Max rY	257	4:1.4DL+1.4SI	5.737	-0.060	-0.160	5.740	-0.000	**0.001**	-0.001
Min rY	256	4:1.4DL+1.4SI	-5.737	-0.060	-0.160	5.740	-0.000	**-0.001**	0.001
Max rZ	143	4:1.4DL+1.4SI	1.701	-56.822	0.813	56.853	-0.004	0.000	**0.007**
Min rZ	137	4:1.4DL+1.4SI	-1.701	-56.822	0.813	56.853	-0.004	-0.000	**-0.007**
Max Rst	77	4:1.4DL+1.4SI	-0.000	-90.720	1.852	**90.739**	-0.004	0.000	-0.000

TABLE 5.3: AISC Deflection Values

The red-colored circle in both the figures 5.9 & figure 5.10 is the maximum encountered area of the deflection. Note from the analysis that the value of the deflection check varies drastically for the two code design models. However, the BS model shows the lower value, which is 74mm (approx.) to that of 110mm (approx.). BS model had the limiting value of deflection within the recommended zone of L/250, which is 75mm, whereas the AISC model has a little higher value of 91mm. As per the design point of view this the BS design has a safer value.

5.5.3 Bending Moment Checks:

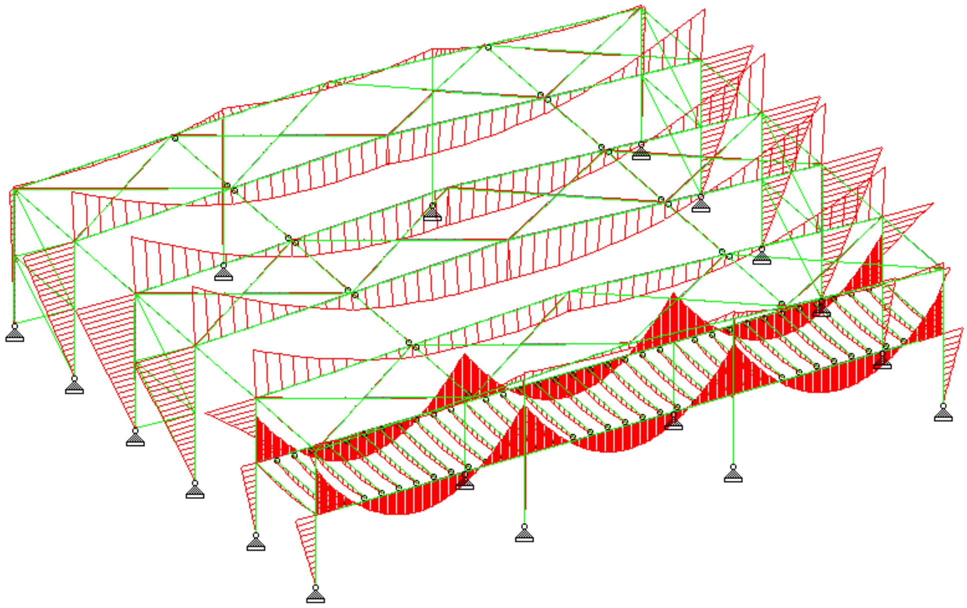


FIGURE 5.11: BS 5950 STAAD Model for Maximum Bending Moment Check

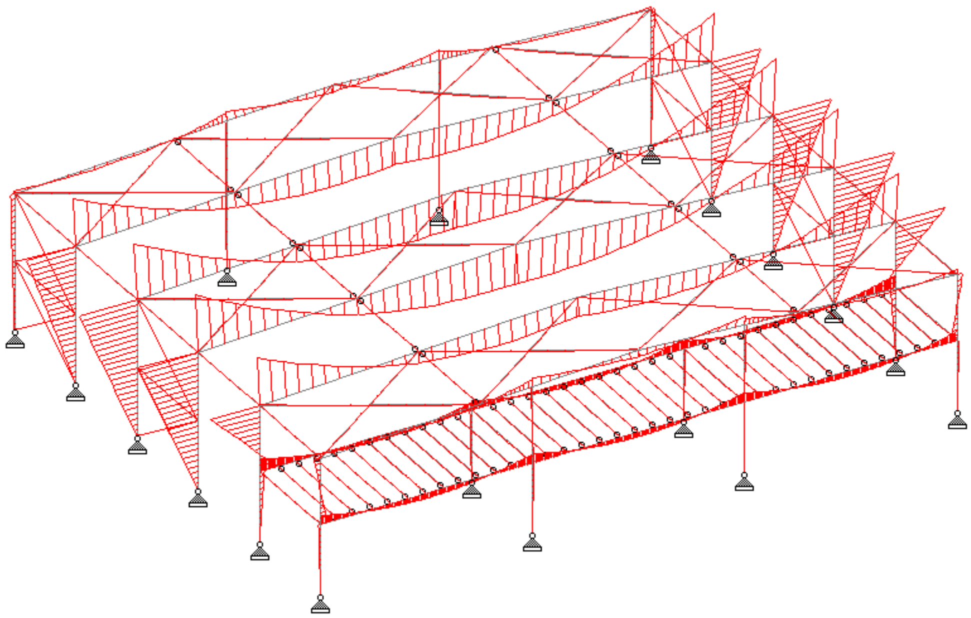


FIGURE 5.12: AISC STAAD Model for Maximum Bending Moment Check

It is evident from the above checks that the bending moment values are higher for the BS model compared to the AISC model. The supporting the analysis table for the bending checks have attached in APPENDIX E, F.

5.5.4 Column Reaction Summary:

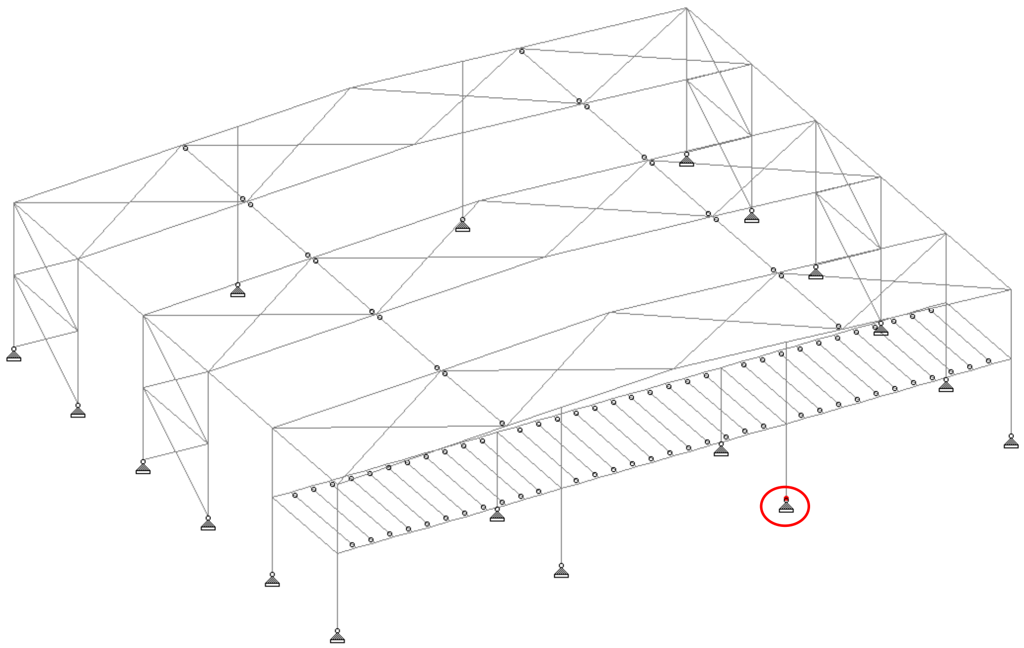


FIGURE 5.12: BS 5950 STAAD Model for Maximum Column Reaction Values

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
Max FX	71	4:1.4DL+1.4SI	45.576	120.746	-16.954	0.000	0.000	0.000
Min FX	73	4:1.4DL+1.4SI	-45.576	120.746	-16.954	0.000	0.000	0.000
Max FY	26	4:1.4DL+1.4SI	1.652	340.832	-0.820	0.000	0.000	0.000
Min FY	73	3:LIVE LOAD	-0.047	-0.297	0.183	0.000	0.000	0.000
Max FZ	1	4:1.4DL+1.4SI	2.981	67.626	15.156	0.000	0.000	0.000
Min FZ	71	4:1.4DL+1.4SI	45.576	120.746	-16.954	0.000	0.000	0.000
Max MX	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000
Min MX	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000
Max MY	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000
Min MY	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000
Max MZ	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000
Min MZ	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	0.000

TABLE 5.4: BS 5950 Column Reaction Values

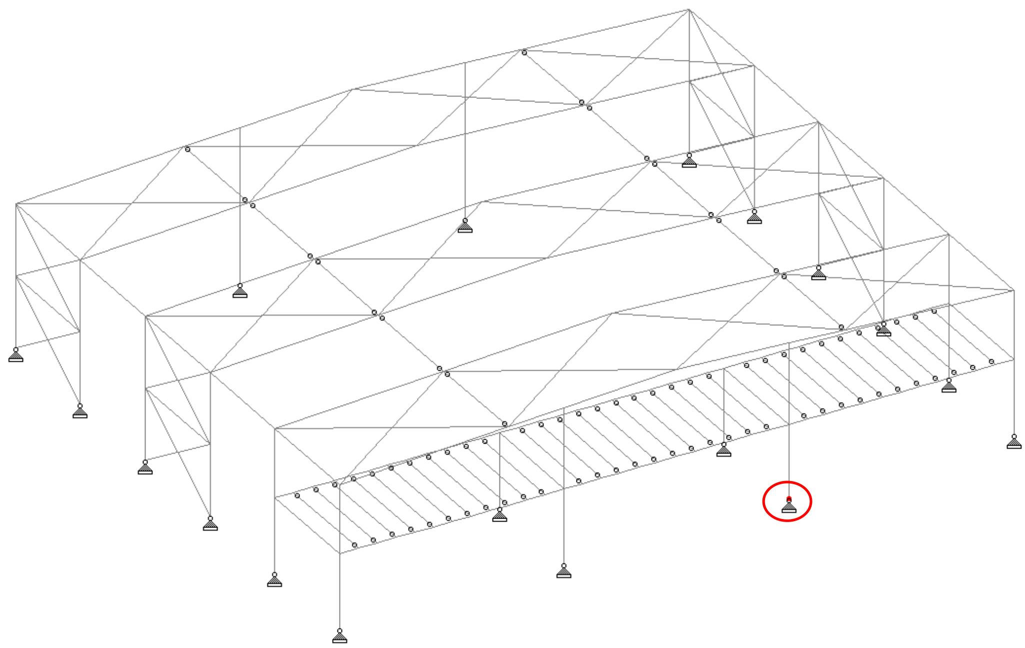


FIGURE 5.13: AISC STAAD Model for Maximum Column Reaction Values

	Node	L/C	Horizontal FX (kN)	Vertical FY (kN)	Horizontal FZ (kN)	Moment MX (kNm)	MY (kNm)	MZ (kNm)
Max FX	71	4:1.4DL+1.4SI	44.013	121.866	-19.237	0.000	0.000	0.000
Min FX	73	4:1.4DL+1.4SI	-44.013	121.866	-19.237	0.000	0.000	0.000
Max FY	25	5:1.2DL+1.2SI	-1.847	300.395	-0.680	0.000	0.000	0.000
Min FY	73	3:LIVE LOAD	-0.050	-0.302	0.189	0.000	0.000	0.000
Max FZ	1	4:1.4DL+1.4SI	2.814	69.887	17.025	0.000	0.000	0.000
Min FZ	71	4:1.4DL+1.4SI	44.013	121.866	-19.237	0.000	0.000	0.000
Max MX	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000
Min MX	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000
Max MY	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000
Min MY	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000
Max MZ	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000
Min MZ	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	0.000

TABLE 5.5: AISC Column Reaction Values

As inferred before in the previous chapters, the column reaction decided the geometry of the steel column base plate and the foundation system to be used. This element is the most critical in the entire structural steel building as it ensures stability; hence, the design of it should be optimum as well as safe. From the computer analysis of both the models, many variations can be observed. Concerning axial, value for AISC seems to be somewhat lower than the BS model; however, for the value of shear in the x-direction, both codes have almost similar values as per load applied. Shear in the z-direction the AISC model is 2kN more than the BS model, which seems to be not that critical.

The values observed in the tables shall be used' for the design of the steel column baseplate with two software's which are:

- Master Series: For the design analysis of the BS 5950 based steel column base plate
- RAM ELEMENTS connection: For the design analysis' of the AISC based steel column baseplate

Different software used due to the limitation of the availability of the two codes. However, the results yield to be approximately accurate in both cases.

The STAAD analysis for both the BS and the AISC models full report shall be attached in the APPENDIX E and F for further insight.

5.6 Steel Column Base Plate design:

5.6.1 Design as per BS 5950:

The following output of the design analysis is with respect to the BS 5950.

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

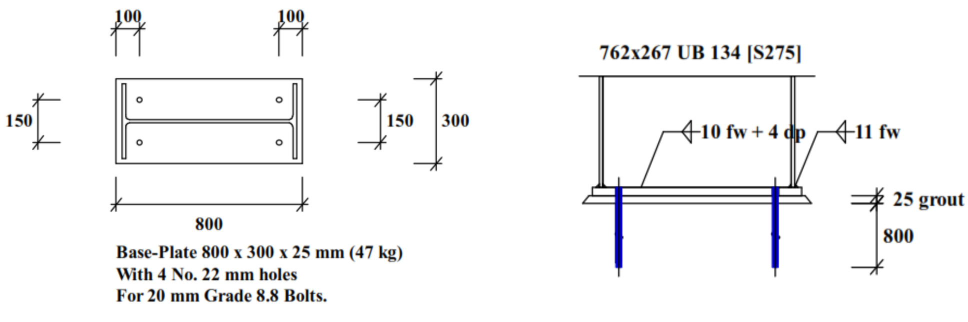


FIGURE 5.14: Geometric considerations for BS 5950

MEMBERS

<u>Column</u>

Column type	:	Prismatic member
Section	:	UB 762x267x134
Material	:	S275

Longitudinal offset : 0 mm

DEMANDS

Description	Pu [KN]	Mu22 [KN*m]	Mu33 [KN*m]	Vu2 [KN]	Vu3 [KN]	Load type
CASE1	340.832	0.00	0.00	45.576	15.156	Design
CASE2	-0.297	0.00	0.00	-45.576	-16.954	Design

The most critical case have been used for the analysis of the

Applied Forces at Interface

Resultant Forces M, Fv, F	Moment +0.0 kNm, Shear +45.5 kN, Axial +340.8 kN (Axial Compression)		

Basic Dimensions

Column: 762x267UB134 [28]	D=750.0, B=264.4, T=15.5, t=12.0, r=16.5, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275, Welds E 35			
Data grout, Fcu, Fcv, py, slope	15 N/mm², 40 N/mm², 0.35 N/mm², 265 N/mm², 30 deg to vertical		
Design to	BS 5950-1: 2000 and the SCI Green Book: Joints in Steel Construction : Moment Connections: SCI-P-207/95		
Column Capacities Mc, Fvc, Fc	1277.2 kN.m, 1485.0 kN, 4691.0 kN	Fc = 4691.0 kN	OK

Summary of Results (Unity Ratios)

Concrete Pressure		1.00	OK
Base-Plate thickness in Compression		0.10	OK
Horizontal Shear		0.35	OK
Flange & Web Welds	0.02	0.02	OK

Step 1: Base-Plate Pressure

Allowable Pressure=0.60•Fcu	0.60•15	9.0 N/mm²	
Base ecc=M/F	0.0/340.8	0.0 mm	
Pressure Configuration	Compression Only with Optimised Compression Area		
Proj, Xcc	8.2, 400.0		
L Zones X1, X2, X3, X4, X5	16.8, 31.9, 702.6, 31.9, 0.0		
W Zones Wstiff, Wflange, Wweb	0.0, 280.8, 28.4		
Ac=x2•wf+x3•ww+x4•wf	31.91•280.81 + 702.59•28.41 + 31.91•280.81	378.8 cm²	
Conc Cap C=0.60•Fcu•Ac	0.60•15•37877.0	340.9 kN	OK
Pressure=P•1000/Ac	340.8•1000/37877	9.00 N/mm²	OK

Step 2a: Plate Compression Bending

e=L1	8.2	8.2 mm	
$M_{app}=p{\bullet}e^2/2$	$9.0{\bullet}8.2^2/2$	303 Nmm/mm	
$tp=\sqrt{(6{\bullet}M_{app}/p_y)}$	$\sqrt{(6{\bullet}303/265)}$	2.6 mm	OK
Note: Axial Load Axial	Using Elastic Modulus Zp (4.13.2.2)		

Step 4: Shear

Base Friction

Friction Fr=0.30•Fc	0.30•+340.8 kN	102.2 kN	

Bolt Bearing

Bolt Shear Bs=Fn(ShrCap, tg)	91.9, 50	91.9 kN	
Concrete Bearing Cb=3•Ø²•0.4•Fcu	3•20²•0.4•15 (No Shear Reinf.)	7.2 kN	
Plate Bearing Pb=Fn(pb,e,Ø,t,kbs)	460, 100, 20, 25, 1.00	230.0 kN	
Bolt Bearing Bb=pb•Ø•tk	1000•20•25	500.0 kN	
Pss=Min(Bs, Cb, Pb, Bb)•nbs	Min(91.9, 7.2, 230.0, 500.0) = 7.2•2	14.4 kN	
Pts=Min(Bsten, Cb, Pb, Bb)•nbt	Min(91.9, 7.2, 230.0, 500.0) = 7.2•2, (no tension)	14.4 kN	

Total Shear Capacity

Total Cap=Fr+Pss+Pts	102.2 + 14.4 + 14.4	131.0 kN	OK

Step 5: Flange & Web Welds

Load dispersal	Flanges resist Moment and Axial, Web resists Axial and Shear. Direct Bearing therefore design for tensile forces only.		
Areas A, Af, Aw	170.6, 2 x 41.0, 86.3 cm²		

Flange Welds

Fapp=F•Af/A	340.8•41.0/170.6 No Resultant Tensile Force	0.0 kN	

Web Welds

Web weld load=Fv/(D-2(fw+T))	45.5/(750.0 - 2(11 +15.5))	0.07 kN/mm	
Fcap w=2•0.7•leg•Py	2•0.7•14•220	4.31 kN/mm	OK

steel column base plate.

5.6.2 Design as per AISC:

The following output of the design analysis is with respect to the AISC.

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

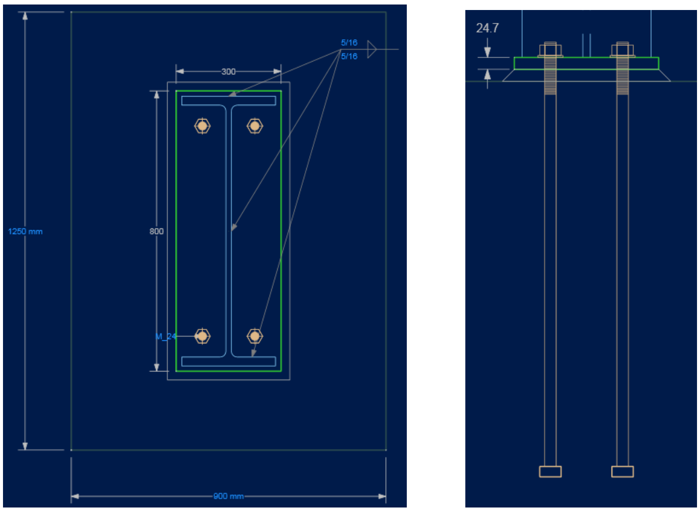


FIGURE 5.15: Geometric considerations for AISC

MEMBERS

Column		
Column type	:	Prismatic member
Section	:	UB 762x267x197
Material	:	S275
Longitudinal offset	:	0 mm

CONNECTOR

Base plate		
Connection type	:	Unstiffened
Position on the support	:	Center
N: Longitudinal dimension	:	800 mm
B: Transversal dimension	:	300 mm
Thickness	:	24.7 mm
Material	:	S275
Column weld	:	E70XX
Outer welds flanges only	:	No
D: Column weld size (1/16 in)	:	5
Override A2/A1 ratio	:	No
Include shear lug	:	No
Support		
With pedestal	:	No
Longitudinal dimension	:	1250 mm
Transversal dimension	:	900 mm
Thickness	:	1000 mm
Material	:	C50-60
Include grouting	:	Yes
Grout thickness	:	25 mm
Anchor		
Anchor position	:	Longitudinal position
Rows number per side	:	1
Anchors per row	:	2
Longitudinal edge distance on the plate	:	100 mm

Transverse edge distance on the plate	:	75 mm
Anchor type	:	Headed
Head type	:	Square
Include lock nut	:	No
Anchor	:	M_24
Effective embedment depth	:	800 mm
Total length	:	881.38 mm
Material	:	A36
Fy	:	0.248 kN/mm2
Fu	:	0.4 kN/mm2
Cracked concrete	:	No
Brittle steel	:	No
Anchors welded to base plate	:	No
Anchor reinforcement		
Type of reinforcement	:	Primary
Tension reinforcement	:	No
Shear reinforcement	:	No

Design code: AISC 360-10 LRFD, ACI 318-08

DEMANDS

Description	Pu [KN]	Mu22 [KN*m]	Mu33 [KN*m]	Vu2 [KN]	Vu3 [KN]	Load type
CASE1	300.40	0.00	0.00	44.01	17.03	Design
CASE2	-0.30	0.00	0.00	-44.01	-19.24	Design

Design for major axis

Base plate (AISC 360-10 LRFD)

GEOMETRIC CONSIDERATIONS

Dimensions References	Unit	Value	Min. value	Max. value	Sta.	
Base plate						
Distance from anchor to edge	[mm]	63.00	6.35	--	✔	
Weld size J2.4	[1/16in]	5	4	--	✔	table

DESIGN CHECK

Verification References	Unit	Capacity	Demand	Ctrl EQ	Ratio	
Concrete base						
Axial bearing 3.1.1;	[KN/mm2]	0.04	0.00	CASE2	0.00	DG1
Base plate						
Flexural yielding (bearing interface) Sec 3.1.2	[KN*m/m]	37.75	0.00	CASE2	0.00	DG1
Flexural yielding (tension interface) Eq. 3.3.13	[KN*m/m]	37.75	37.55	CASE1	0.99	DG1
Column						
Weld capacity 9, J2.5, J2.4, p. 35	[KN/m]	1828.47	631.08	CASE1	0.35	p. 8- Sec. Sec. DG1
Elastic method weld shear capacity 9, J2.5, J2.4	[KN/m]	1218.98	32.08	CASE1	0.03	p. 8- Sec. Sec.
Elastic method weld axial capacity 9, J2.5, J2.4	[KN/m]	1828.47	308.16	CASE1	0.17	p. 8- Sec. Sec.
Ratio	**0.99**					

Major axis
Anchors
GEOMETRIC CONSIDERATIONS

Dimensions References	Unit	Value	Min. value	Max. value	Sta.	
<u>Anchors</u>						
Anchor spacing	[mm]	150.00	96.00	--	✓	Sec. D.8.1
Concrete cover	[mm]	313.00	76.20	--	✓	Sec. 7.7.1
Effective length	[mm]	815.60	--	984.40	✓	

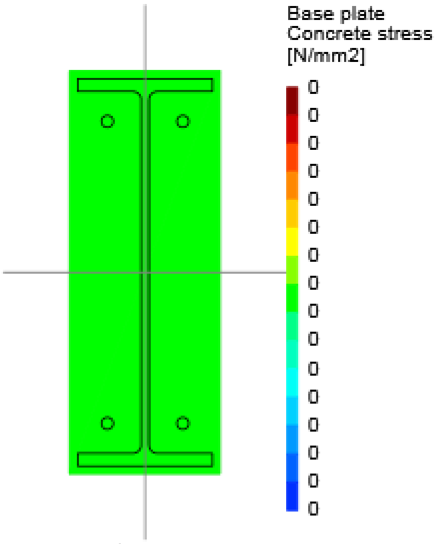
DESIGN CHECK

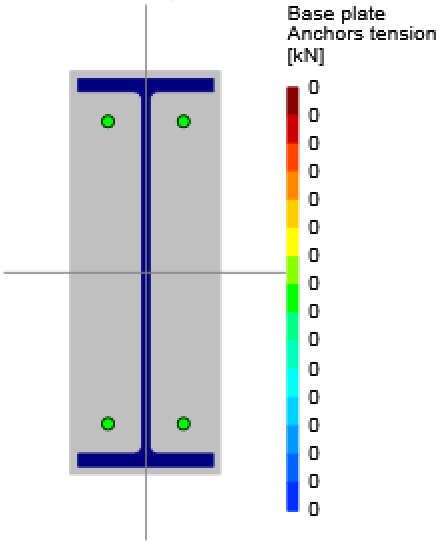
Verification References	Unit	Capacity	Demand	Ctrl EQ	Ratio	
Anchor tension	[KN]	102.96	75.10	CASE1	0.73	Eq. D-3
Breakout of anchor in tension	[KN]	285.80	75.10	CASE1	0.26	Eq. D-4, Sec. D.4.1.1
Breakout of group of anchors in tension	[KN]	471.62	300.40	CASE1	0.64	Eq. D-5, Sec. D.4.1.1
Pullout of anchor in tension	[KN]	330.85	75.10	CASE1	0.23	Sec. D.4.1.1
Anchor shear	[KN]	42.83	11.00	CASE1	0.26	Eq. D-20, Sec. D.6.1.3
Breakout of anchor in shear	[KN]	144.28	11.00	CASE1	0.08	Sec. D.4.1.1
Breakout of group of anchors in shear	[KN]	188.23	44.01	CASE1	0.23	Sec. D.4.1.1
Pryout of anchor in shear	[KN]	571.59	11.00	CASE1	0.02	Eq. D-4, Sec. D.4.1.1
Pryout of group of anchors in shear	[KN]	943.24	44.01	CASE1	0.05	Eq. D-5, Sec. D.4.1.1
Interaction of tensile and shear forces	[KN]	1.20	0.99	CASE1	0.82	Eq. D-3, Eq. D-4, Sec. D.4.1.1, Eq. D-5, Eq. D-20, Sec. D.6.1.3, Eq. D-32

Ratio	**0.82**

Global critical strength ratio	**0.99**

Major axis
Maximum compression (CASE2)



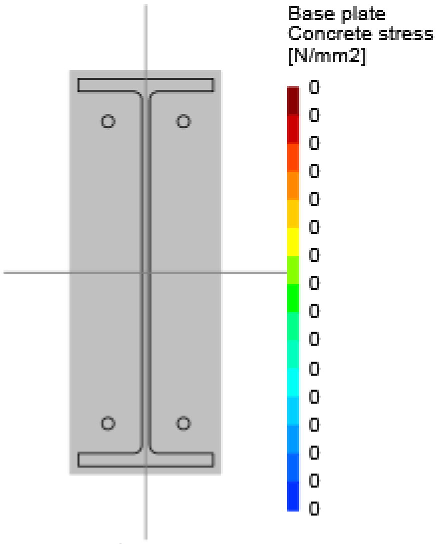


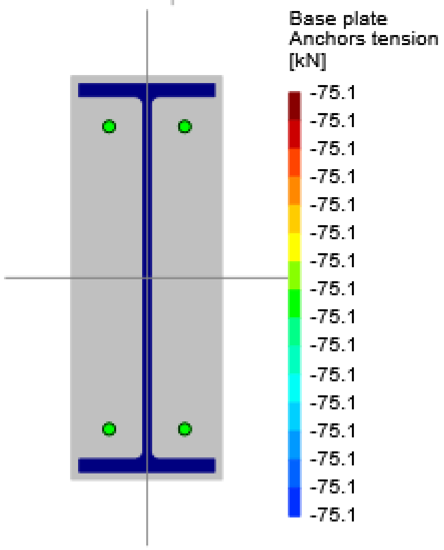
Maximum bearing pressure	0.00	[N/mm2]
Minimum bearing pressure	0.00	[N/mm2]
Maximum anchor tension	0.00	[KN]
Minimum anchor tension	0.00	[KN]
Neutral axis angle	0.00	
Bearing length	1E33	[mm]

Anchors tensions

Anchor	Transverse [mm]	Longitudinal [mm]	Shear [KN]	Tension [KN]
1	-75.00	-300.00	-11.00	0.00
2	-75.00	300.00	-11.00	0.00
3	75.00	300.00	-11.00	0.00
4	75.00	-300.00	-11.00	0.00

Maximum tension (CASE1)





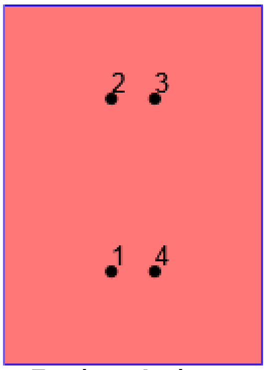
Maximum bearing pressure	0.00	[N/mm2]
Minimum bearing pressure	0.00	[N/mm2]
Maximum anchor tension	75.10	[KN]
Minimum anchor tension	75.10	[KN]
Neutral axis angle	0.00	
Bearing length	-1E33	[mm]

Anchors tensions

Anchor	Transverse [mm]	Longitudinal [mm]	Shear [KN]	Tension [KN]
1	-75.00	-300.00	11.00	75.10
2	-75.00	300.00	11.00	75.10
3	75.00	300.00	11.00	75.10
4	75.00	-300.00	11.00	75.10

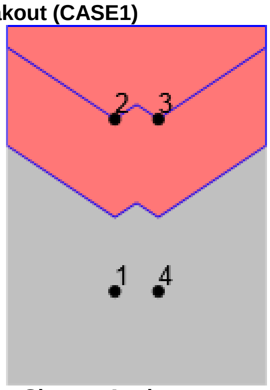
Major axis

Results for tensile breakout (CASE1)



Group	Area [mm2]	Tension [KN]	Anchors
1	1125000.00	300.40	1, 2, 3, 4

Results for shear breakout (CASE1)



Group	Area [mm2]	Shear [KN]	Anchors
1	900000.00	44.01	1, 2, 3, 4
2	438750.00	22.01	2, 3

5.7 Discussions and Recommendations:

The section above in the chapter is the critical comparison intended for the code BS 5950 and AISC for the steel building design codes. The output henceforth was compared for an insight that might prove more economical along with the safety consideration of the structural elements. The scope of the study was over the design parameters of the model like utilization ratios, the deflections of the model, bending moment, the column reaction summary, and the overall economic weight of the structure. It would provide a better understanding of the design codes used in this work with aspect to the country of U.A.E.

The comparison was made to check the similarities and disparities among the codes. However, the following are the significant observations from the analysis:

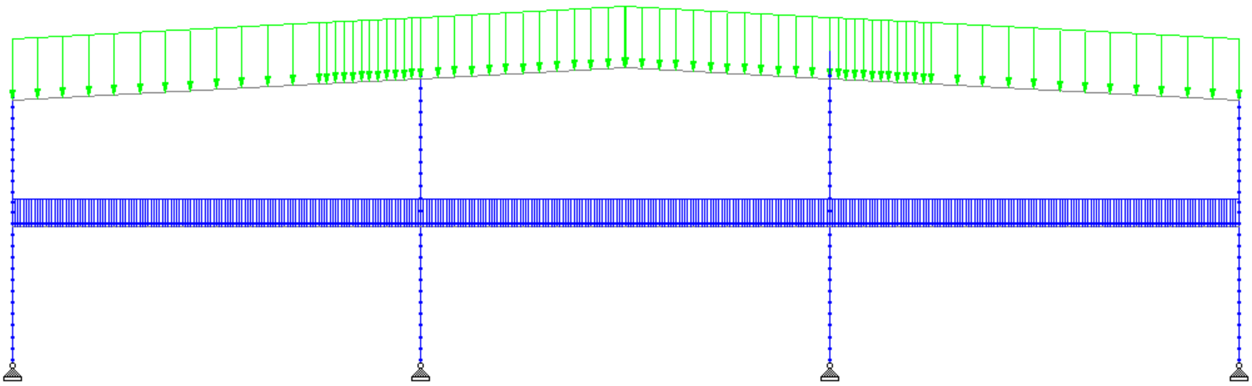


FIGURE 5.16: Load Path of the Structure

- The loads applied to the structure as per the code recommendation. The applied areas where the roof and the first floor of the building as it was the critical areas in the model. The load path ultimately ends at the bottom of the column which pinned base connection where the loads coming from the roof and the first floor flows parallel to the gravity coinciding at the bottom of the column end.

- As the factored loads considered in the model as per the code recommendations, the load statics varied in both models resulting in the column reaction values to differ. The more axial force value was for the B.S. model and the shear value for the AISC model.
- Noted that the bending moment profoundly differed among the codes as observed from the analysis results, which is the value for the B.S. model was much higher than the AISC model. However, the deflection as well as comparatively more to the AISC model. It shows that the AISC code has more checks and ensures all elements to pass these checks, whereas the B.S. model checks are fundamental and critical.
- The column reaction values were used to design the steel column base plate as per the codes, and again, observed that a similar approach used which is the difference of several checks. These checks comply with those who have discussed in the previous chapters. The geometric values of the base plate were higher in the AISC model with 24mm bolts as for the B.S. model was 20mm, but the base plate thickness was lower than the B.S. design which seems to be insignificant.
- As in theory, the AISC code also gives an insight into the concrete foundation as well, whereas the BS 5950 has a basic bearing check involved.

From this observation, it is clear that the British standards would provide an industrial design that is safe but the AISC design codes would ensure more progress with much safer elements but compromising the financial budget of the assembly. The AISC design might prove to have more life span compared to the B.S. designs. It concludes from the discussion that the inferences drawn from the analysis almost do not agree to the theory derived in the earlier chapter since the AISC showed to have the upper hand than the B.S. design of the international steel building design codes. However, the design of the entire structure seems to be much safer and the weight seems to have reduced in the AISC model. Considering the overall project, the recommendation is AISC would have a much safer and cost-effective approach. The detailed output of the design analysis for both BS 5950 and AISC would be available in APPENDIX E, F for further study.

However, this criterion depends on the contractor or the client to choose the economic deal and how long they decide the building constructed. In U.A.E., as discussed before the B.S. design code has the upper hand in almost any consultant. If there may arise a conflict between the choosing of the standards and the codes, the engineer shall approve those methods as per the most stringent conditions encountered.

CHAPTER 6: CASE STUDY ON THE STEEL COLUMN BASE PLATE AND THE ADVANCEMENTS

6.1 Case Study:

6.1.1 Agenda:

This case study is intended to understand the connection among the concrete footing and the steel column, which is the fixation of the anchor bolt connections. It is to bring about why the J-type or L-type anchors are still relevant in the country of U.A.E. and concur with the design codes intended in the region.

6.1.2 Introduction:

The area for the case study based in Dubai, U.A.E. The structure intended to build is a G+M Steel Warehouse where the column and beam profiles used are pre-engineered fabricated section sizes rather than using the ready-made standard Hot rolled sections like U.B., UC, W-sections. The size of the warehouse is approximate 37m X 28m X 9m (L X B X H). The First option contracting L.L.C handles the construction of the structure. The plot No. is 2848748, Al TTAY, Dubai. The site is under the supervision of Er. Leo Ramos. The steel supplier of the building is Asia Bolts Industries L.L.C supplies roofing Middle East steel construction L.L.C and the bolts.

6.1.3 Brief:

As the country isn't in the seismic zone the standard type of anchor bolts used in the region are the L-type, J-type, and cast in place anchor bolts (anchor bolts with 5mm plates at concrete side). Only when the structural forces demand chemical anchor bolts used. The L-bolts used when the column reactions are considered to be in the normal range; then the J-bolts are used when the reaction occurred during the design analysis considered in the medium range. For those forces from column reaction which are more than a medium-range that is high, that's when cast in place bolts used.

Usually, the material chosen for the anchor bolts are 4.6 grade S275Jr but in exceptional cases like massive column reactions bolt which 8.8 grade S275JR used. The bolts usually painted with red oxide wherein the minimum thickness is kept at 150 microns. If it is required the anchor bolts are hot-dipped galvanized just like in the structure carried out for the case study. The bolts may be galvanized on its threaded area which is projecting above the concrete footing or in some cases the entire bolt is galvanized. This process is entitled to prevent corrosion in the anchor bolts.

The anchor bolt fabricated as per the required design dimensions, and the threads cut accordingly. These anchor bolts then subjected to sand or grit blasting to remove any rust after which the bolts immediately painted with red oxide primer to prevent corrosion. It is a painting process to protect the anchor bolts. The other process is by galvanizing the anchor bolts. They are cleaned in an acid bath to remove any rust, cleaned with steel wired brush and then coated with zinc as per requirement.

6.1.4 Observations:

The area of the site for G+M Warehouse has excavated to an intended level which is above 500mm from ground level for the preparation of the construction of foundations to maintain footing above the finish floor level. The column' baseplate and the concrete footing of a warehouse were connected using the anchor bolts. The setup already fixed with wooden shuttering and wooden template.

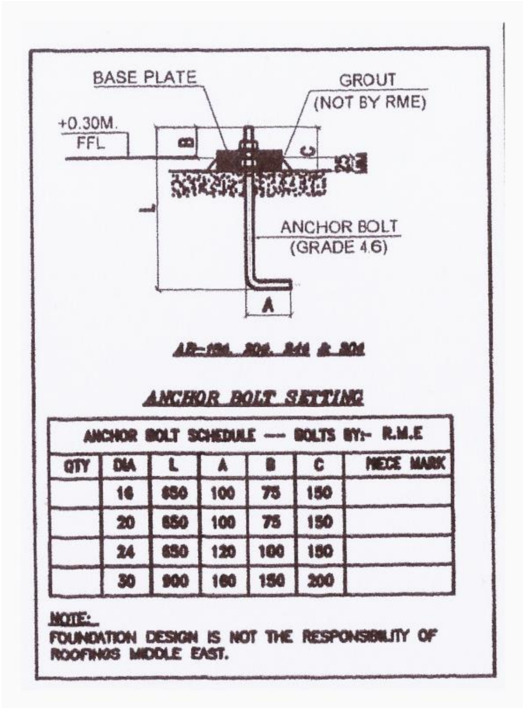
6.1.5 Investigation:

The level has compacted, and three in site density tests and proctor tests were carried out as per the soil report provided for the warehouse zone, where all the tests indicated that the formation level had been to an average of 96% M.D.D. (maximum dry density). The conditions of the sidewalls indicated consistent with those in the boreholes.

For the footing the allowable bearing pressure up to 175kN/m2 for isolated pad footing and 125kN/m2 for strip footing where intended, set on the top of the above-compacted level. The type of anchor bolt used to connect the steel column base plate to the concrete footing was the L-type anchor bolts.

The specifications for the L-type anchor bolt used in the warehouse are as follows:-

- The material used for the anchor bolt is ISO 898-1, and the grade chose for it is GR.4.6.
- The size intended to use for the anchor bolts is M20 X 580.
- The tensile strength for the anchor bolt is specified to be 431 N/mm2 with hardness as 84HRB
- The yield strength for the bolts are allowed to be 285 N/mm2 with elongation up to 33%
- The type of nut used where the hex nuts where the material is DIN 267 with grade CL8
- The size used for the nut is M20 itself with hardness 29HRC and proof load 188 kN
- The surface finish of the anchor bolts where hot dipped galvanized as per ASTM A153 CL C.
- The dimensions of the anchor bolts given in the figure below.



ANCHOR BOLT SCHEDULE --- BOLTS BY:- R.M.E						
QTY	DIA	L	A	B	C	PIECE MARK
	16	650	100	75	150	
	20	650	100	75	150	
	24	650	120	100	150	
	30	900	160	150	200	

FIGURE 6.1: The dimension as per the design for the structure of G+M Warehouse

6.1.6 Methodology:

- After locating the foundation, weld four vertical steel rods onto the four corners of the steel formwork. The length of the vertical steel bars depends on the length of the anchor bolts.
- Ask the surveyor to mark the required concrete height on the newly welded vertical steel rod and apply masking tape to demarcate the height.
- The center of the anchor bolts marked with the help of a Theodolites by checking all the horizontal, vertical, and diagonal dimensions along with the appropriate grid location marks.
- Weld 4 horizontal steel rods, to form a square at the concrete height.
- Get the surveyor, to locate the center of the 4 horizontal rods; mark center with an encircled dot.
- Tie line dories strings connecting the centers of opposite horizontal rods; these would be the center lines of the template.

- Tack weld two horizontal steel rods parallel to any one of the line dories strings, as support to tack weld the template.
- Use a crowbar, to deform the steel rods within the cage to facilitate the fixing of the anchor bolts in place. Cut rods if required to avoid slanting of anchor bolts.
- Insert the anchor bolts into the cage and align them with the holes in the template.
- Place the template on top of the two support rods, coincide the centerlines marked on the plate with the line dories strings & tack weld the plate onto the support rods.
- As per the case study conducted, the anchor bolts fixed with the help of a wooden template. All the templates were nailed to the wooden shutter, as shown in the picture. It is purely done to avoid the movement of the anchor bolts while pouring the concrete and then after the vibration has done to remove the air pockets from the concrete. The anchor bolt is tied to the reinforcement in the concrete as well.
- Tighten the anchor bolt nuts to hold the template in place.
- The surveyor must inspect the coordinates of the top of each anchor bolt, to ensure that it is in the right position.
- Then the after placement of the concrete and then after curing of it, the wooden / steel template is to be removed along with the wooden shutters provided.
- A minimum of a 25mm gap is to be maintained while erecting the column as shown in the picture. This gap provided between the steel column' baseplate and the concrete' footing is to fill it with grout. The grout used in is a non-shrink type. The grouting would thus help to maintain the level of steel columns as well as the effective distribution of loads.
- After completing the steel erection, checking the alignment and torquing the bolt by the specified forces prescribed the designer, pour the grout under the steel base plate to fill in the provided gap using a proper wooden shuttering which made for the process all around the base plate.

- After the completion slabs may require the anchor bolts fixed with machines. If there is a possibility for future expansion chemical or mechanical rawl bolt anchoring is recommended as per the design force required.

6.1.7 Precautions:

- It is to check in the zone of constructions for any vibrating equipment which may interfere with the anchor bolt apparatus. The vibration may cause the anchor bolts to loosen up and interfere with the alignment of the mechanisms. If such machines used a double nut is to be added to the anchor bolts and torque the anchor bolt with the designated forces. It would make sure the bolts are in the intended place.
- Clashes between the anchor rods and the reinforcement to be checked by the surveyor, if found the tie wires aligning the anchor bolts need to untie the steel wire and arrangement need to be rectified accordingly to accommodate the bolts with the reinforcement. Then the tie backs the steel wire that keeps the anchor bolts in the group.
- If the anchor bolts damaged in the process of erection or transportation, it is recommended to rectify the shape to its original state using an apt tool before the placement or erection of the steel columns. It is critical not to damage the threads of the anchor rods while amending. If possible, use a new anchor rod.
- The anchor bolts must be kept clean from dirt and other physical impurities. It can be achieved by taping heads and below side as well as an exposed area with tape as shown in the picture.
- Fixing the anchor bolts is to be done during good weather to avoid mistakes. Do not commence the fixation of the bolts during dusty, heavy wind and foggy weather conditions as it obstructs the view while fixing the intended.
- The anchor bolts have to check for proper galvanizing by randomly choosing any anchor bolt and scratching it as some part of it would be duplicated by painting the same color.

- The same grade followed for both the bolts and the base plate; otherwise, the apparatus may react to corrosion.

FIGURE 6.2: L-Type Bolts Used In the G+M Warehouse

FIGURE 6.3: The Template Marked With Center Lines for Proper Alignment of the Anchor Bolts

FIGURE 6.4: The Wooden Template and the Wooden Shutter with the Anchor Bolt Covered With Tape and Aligned With Reinforcements

FIGURE 6.5: The Gap Maintained For The Non-Shrink Grout To Be Placed Between The Steel Column Base Plate And The Concrete Footing After Erecting The Steel Column.

6.1.8 Conclusions:

According to the observations and investigations concluded the case study is that the project 'G+M Steel Warehouse' used standard L-type anchor bolts which used as a general type of anchor bolt. There wasn't any technologically advanced anchor bolt used in the structure as like any other structure in the U.A.E. since the country is not affected by any earthquake like those countries in the seismic zones like Japan where the drift occurs to break or bending the anchor bolts. It has also noted that the placement of anchor bolts done in harmony with reinforcement of concrete foundation prior to concrete filling. From the investigation it is understood that the priority in any construction is for the L or J bolts as it is readily available, the placement is considered secure and the cost of it is cheaper compared with the other type of anchor bolts. The fabrication and the manufacturing of the anchor bolts purely depend on the structural column reaction based on which the anchor bolts designed and produced. Also observed under every anchor bolt with a template or base plate 25mm of space were left to fill grout which is done after column erection so that the levels are maintained. This part of the base plate anchor bolt apparatus has importance is even transferring the loads. In some of the base plate, grout hole was provided on the base plate to fill the non-shrink flowable grout by using a funnel with confinement as wooden shuttering around the base plate.

6.2 Advancements In Steel Base Plates: Insight To Self-Centering (S.C.) Elements And Shape Memory Alloy (S.M.A.) Anchor Rods

6.2.1 Introduction:

The design philosophy in any seismic resistant structure is to concentrate on those sacrificial structural members who undergo significant inelastic deformation, which is a predetermined region and provide significant energy dissipation. The flow in this philosophy is the cause of severe economic damage to the structure. Hence new concept was proposed, which is self-centering S.C. structural elements with the help of innovations in technologies or materials. Thus achieving maximum damage-free structural systems. The self-centering force provided by those post-tensioned tendons and the use of other devices to add a stable energy dissipation.

The noteworthy side of the system is that it almost eliminates the residual deformation after an event of an earthquake.

Shape memory alloy (S.M.A.) is a new concept that has added to the earthquake community which provides a substitute key to attain a necessary S.C. behavior. S.M.A.'s are a metallic material with high performance to sustain a tremendous strain. However, they can still recoup its initial shape via loading or heating. The characterization of the behavior of the S.M.A. is by the flag like shaped hysteresis loop under cyclic loading, which provides a reliable depiction of the S.C. and energy dissipation competence in the application of seismic protection.

Usual Nickel Titanium is most commonly used in the S.M.A.'s as they have super-elastic behavior and excellent corrosion resistance with suitable fatigue property. Recently a study is being made for the copper' based S.M.A.'s as well due to its superior performance in lesser temperature.

In comparison to other steel connections in a structure, the steel base plate is susceptible to more seismic damages. Observations made by Tremblay (Tremblay, Nakashima, and Midorikawa, 2018) confirms this fact were the damages include the fracture in the weld, crushing of grout or concrete under the base plate, either fracture or elongations in anchor rod and yielding of the base plate under a load of action. Thus innovation has been exercised to facilitate a damage-free column base connection. The quantity of the S.M.A. bolts used in a baseplate connection can be modified to suit the targeted flexural strength of the column. The super-elastic nature of S.M.A. bolts controls S.C. (self-centering) capacity under cyclic' loading by permitting the base to have a rocking behavior and give moderate dissipation' of the produced' energy through a flag like a loop which is supposedly a hysteresis loop. As per the proposed design initiated the S.C. column is to retract back to its preliminary position with very minimal almost to nothing residual drift. In a building, the target story drift ratio is interrelated to the superlative recoverable base moment. The ratio occurred of the drift can be resolved by a system of analyzing the structure' globally with different levels of intensity of seismic in consideration.

6.2.2 Properties of Super-elastic S.M.A (Mechanical):

The superelasticity of S.M.A. is the effect of the stress-induced change in the phase at a temperature over the austenite'' finish temp. 'A_f' and is linked recovery' with' strain after unloading. The following figure shows a method' of stress– strain relationship of super-elastic S.M.A. The austenite' which would undergo elastic'' loading 'o' to 'a' when applied with an original force' is. Plateau type which is flat that are succeeding from 'a' to 'b' is showed in phase forward change of conversion from' austenite' over to martensite, as well as the "yield-like" stress 'σ_{Ms}' signifies the forward' onset' of a change. After the end of' transformation, the stress level 'σ_{Mf}' is implied by a' distinctive increment in the slope' of the stresses– 'strain' association. Successive strain solidifying speaks to the loading of elasticity conduct of' martensite 'b' to 'c.' The earliest path which has been unloading is almost straight, and the mechanical' forces are discharged bit by bit to stress level 'σ_{As}' 'c' to 'd'. The plateau which is unloading 'd' to 'e' with similar early width as that of plateau level which is loading from 'b' to 'c' can be seen during' the retrogressive phase' conversion' from martensite' over to' austenite. After' the phase finishes the retrogressive' transformation' at stress' level 'σ_{Af},' the path of from 'e' to 'f' described by the elasticity' of 'the' austenite. The process of unloading' and loading stage, S.M.A. exhibits' a flag molded hysteresis loop in with a slight' leftover strain' 'ε_{res}' and a release' of the moderate capacity of energy, which portrays to a satisfactory feature' from the' viewpoint of seismic design engineering.' Additionally, nothing like the yielding' characteristics of' steel, the stress plain of S.M.A. is the' result of the phase' transformation by distortion through shear' lattice and doesn't collaborate with any induced' damage in' the S.M.A.

To’ avoid a relaxation from the unwinding of the S.M.A. bolts because of the collected remaining strain’ under’ cyclic loading, a pre-strain ε_{pre} of’ the S.M.A. bolts suggested. Such pre-strain level influences the behavior of the column, for example, strength by base decompression, forces developed by restoring, and deformation at the residual. As the S.M.A.'s of ultimate deformation and stiffness’ degradation while the phase’ changes, the level’ of pre-strain in the S.M.A. bolts’ should be’ restricted as $\varepsilon_{res} < \varepsilon_{pre} < \varepsilon_{Ms}$.

After the cycle of loading-unloading, the initial pre-stress stage possibly’ or might’ diminish from point o′ to point f′, where the separation among the’ two’ represents a conceivable pressure misfortune σ_{lost}. The pre-stress loss reimbursed by fixing the bolts tightly after a seismic tremor only if it’s necessary.

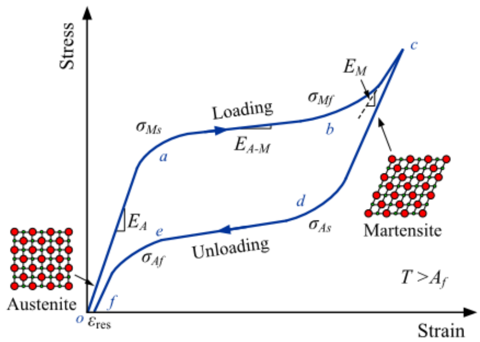


FIGURE 6.6: Typical Mechanical Behavior of Superelastic SMA’s (Tremblay, Nakashima, and Midorikawa, 2018)

6.2.3 S.C. Column’s behavior subjected to Cyclic Loading:

Reactions of an S.C. columns’ with S.M.A. bolts under cyclic’ loading relies upon two essential components which are S.M.A. (shape memory alloy) bolts and the compressive’ force by axial. To describe’ the base’ moment vs. top’ drift in ratio reaction of an S.C. column with S.M.A. bolts, many key’ states of the limit attained by the strain’ dimensions of the’ S.M.A. bolts. The’ idealized reaction of the moment at base ‘Mb’ versus that of a drift’ ratio ‘θcol’ for’ the S.C. column, with the sign of the descriptive states at the limit.’ The included type of conditions is - the base plate decompression, the beginning of a transformation in forwarding phase in the

S.M.A. bolts which is represented by a yield like a point, the maximum point of recovery which compares to the transformation of the consummation of phase in the S.M.A. bolts and the max. Recoverable super-elastic disfigurement and ultimate point finally by which any loading any further might cause potential break/rupture' of S.M.A. bolts. The structure of S.M.A. bolts depends on the ultimate point.

From beginning point which is from 'O' to point 'A', the' column disfigurement straightly until the point of' decompression of baseplate when the moment which is overturning causes the lateral' loads overcomes the moment resistance is given by original' post-tension in the S.M.A. bolts as well as the compression through axial in a column. In this phase of loading, no shaking conduct in the column' seen, and all the S.M.A. bolts' about keep up the condition of primary' pre-strain. Therefore, for that of an initial stiffness' of a column's' like one with' a wholly fixed connection to a column end base. After' the process of decompression' a breach opening' extents' along' flat interface at the column base end with the amassed revolution in' respect to the right edge under steady lateral' load. As the end of a column's base elevates, the S.M.A. bolts' stretch and the deformation in the bolts (elevated side) end up more significant than the ones in the' right. The bolts over to the left start to encounter the austenite' phase change over to martensite phase change firstly' and display a yield-like state. The correct bolts experience a small amount of twisting than the left' bolts and are accepted to disfigure basically in the state of' austenite' amid the entire phases of applied' loadings. As there is that gap opening which expands, the left bolts total a change' in martensite.

Amid unloading, the' S.M.A. bolts ' and forces cause of the compressive by axial give restoring forces to come back in the column' to its initial position' – S.C. conduct. At the same time, the S.M.A. bolts disseminate energy utilizing the inherent hysteresis loops which shows a flag like shaped as discussed above. Amid the' unloading at the primary stage, stiffness during the stage of unloading' is considered to be almost the same. When' unloading proceeds, the left S.M.A.

bolts return to austenite from the martensite phase’ continuously. At last, the column recuperates its original’ state after the elastic distortion of the column. Amid the entire cyclic’ loading, the S.C. column displays a run of the flag’ shaped hysteresis conduct with negligible of residual drifts. At the point when forces along the lateral’ connected to the direction which’ is opposite, edge towards’ left of baseplate turns into the center of rotation, and overturning’ moment– drift’ ratio reaction of an’ S.C. columns in symmetry.

CHAPTER 7: CONCLUSION

7.1 Conclusion of Research:

The primary intention of this work was to bring about the comparison between the principal international codes, namely, European, American, and British Standards through literature review. However, considering the relevance of the emphasized codes in U.A.E. the study was limited to two codes which were the British Standards and American Standards which then compared with the help of output results obtained from the computer analysis and the design of a steel warehouse. Initially, a general behavioral pattern of the steel base plate and the members existing in the assembly described for the approach of design and how the chosen code would affect the steel base plate assembly. Then followed by the parametric study concerning each code with empirical formulae to compute the physical quantities of the steel column base plate. As discussed both the general comparison and the parametric studies conducted for the three primary international steel building design codes which are EC-3, AISC and BS 5950. The three codes were scrutinized concerning parameters in the relation of variable, permanent, and factored combinations of loads. The course further carried to the comparison with shear, axial, and the moment caused in different scenarios of codes used. However, the British Standards have been phased out from the European region, and the United Arab Emirates emphasizes the British Standards as the majority of the engineers widely choose to work with the design provision of B.S.I. Henceforth comparative study through computer analysis software was focused only on the two relevant codes in the region, namely B.S. and AISC, which is a steel warehouse with only one floor. Even computer analysis was compared based on permanent, variable, and factored combination loads along with shear and axial according to a type of connection provided to column end.

Concerning the interpretations from work, the British Standards preferred over American standards from the design of the base plate point of view. However, note that from a practical point that the analysis of the entire structure of the AISC model had weight lesser than the B.S. model. Since AISC design analysis has several checks and methods than the B.S. design analysis, it can conclude that AISC would prove to be safer and much more economical than the B.S. code as per the computer analysis of the entire structure. This statement viewed as the parametric studies and the other critical conditions like the loads and the load combinations followed in the code aided for its importance slowly in the U.A.E. by considering the design approach as well to be more cost-effective and favorable for a region.

The intended objectives for this work has met, and the results from the computer analysis and the literature review with theoretical findings were carefully examined to provide adequate substantiation to arrive at the primary motive of which code provides the best solution in terms of economy, adaptability, and environment-friendly to the construction industry. This research is aimed to provide a better view for the structural engineers since the U.A.E. is home to construction and a variety of nationals are involved. To maintain uniformity in the design of structures, the country has to maintain a specifically approved code since U.A.E. does not have a complete national design code of its own. In the early eighties, the B.S. code predominantly used, as most of the structural engineers used to the U.K. nationals, which is why the code is so prevalent in U.A.E. However, during the ’90s the American consultants started establishing and practicing the American codes. Henceforth the two design codes are in practice in U.A.E. ever since.

7.2 Scope for Future Research:

The research work was done to compare the three codes namely Eurocode, American, and the British steel design codes and exclusively focusing on the two codes majorly used in the U.A.E. which are AISC and B.S. It was also attempted to bring about the minimum geometrical and the design requirements needed in the method used with respect to shear, axial and the moment as well. The future scope for this research could be the effects of using the nonstandard section profiles, which are the pre-engineered buildings on the steel column base plate design. The comparison can also include the upcoming building codes like Egypt and the Canadian codes. More evidence for the Eurocode can also included in the course of research. This research work deals only with the steel, and it can be extended towards the use of composite structures as well.

Nevertheless, the budget in all cases of construction is always stringent, and it’s up to a structural engineer how efficient the building can be made.

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APPENDICES

APPENDIX A

Loads Considered :-

DEAD LOAD = 1kN/m^2 (DL) , LIVE LOAD = 2kN/m^2 (LL)

EUROCODE-0
U.D.L = 1.35DL+1.5LL
=1.35*1 + 1.5*2 =4.35kN/m^2

ASCE -7
U.D.L = 1.2DL+1.6LL
=1.2*1 + 1.6*2 = 4.4kN/m^2

BS 5950-1
U.D.L = 1.4DL+1.6LL
=1.4*1 + 1.6*2 = 4.6kN/m^2

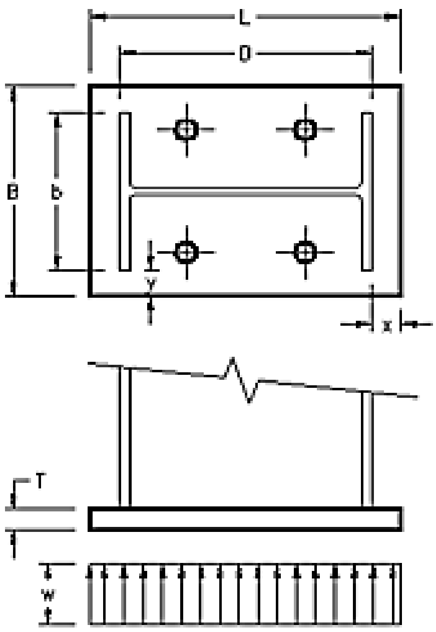
APPENDIX B
BASE PLATE CALULATION DESIGN BS 5950

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

Reactions from analysis

Frame	Node No	Axial, P (kN)	Shear, Fx (kN)	Shear, Fy (kN)	LoadCase
MAIN FRAME TYPE"A"	1	90.00	45.00	0	ULT
Design Forces		90.00	45.00	0.00	

Sections	Column	
Section No	92	
Sec Name	UC203X203X46	
Depth, D	203.20	mm
Flange Width, B	203.60	mm
Flange Thk, T	11.00	mm
Web Thk, t	7.20	mm
DesignStrength,py	275	MPa



DesignStrength for plate, pyp	275	MPa
Assume Thickness of plate	20	mm
Concrete Strength, fc	35	MPa
Permissible bearing pressure, w = 0.6fc	21	MPa

Holding Down Bolts

Bolt Dia, d	20	mm
Bolt Grade	8.8	
No of Bolts, N	4	
Hole dia in Plate, S	22	mm

Effective area, A_{be} > axial load / bearing strength

A_{be} = 4285.71 mm2

Actual Area, A_{be} = (B+2c) (D+2c) – 2 {(D – 2(T + c)) ([B+2c] – [t+2c])}

Using C =	6.7		
4285.71	**>**	**-18909.64**	**OK**

Min Length of plate, L = D+2c =	216.6	mm, say	**220**	**mm**
Min Width of plate, B = B+2c =	217	mm, say	**220**	**mm**

Thickness of Base plate

Plate Overhang along length, x = (L-D)/2 =	8.4	mm	
Plate Overhang along width, y = (B-b)/2 =	8.2	mm	
Permissible Bending stress in plate, Pyp =	275	N / mm^2	
Thk of plate required = c * √((3w/Pyp) =	3.21	mm	Safe
Thickness Provided =	20.00	mm	

Check for Bearing

Resultant Shear = √ (Fx2+Fy2) =	45.00	kN	
Design Shear force, F =	45.00	kN	
Actual bearing stress = F/(S*N*T) =	25.57	N / mm^2	
Permissible bearing strength, Pbs =	460.00	N / mm^2	Safe

Provide Base plate 220 mm x 220 mm x 20 mm Size

Holding Down Bolts

Max Tension	0.00	kN	
Max Shear force, F =	45.00	kN	
Actual Tension / Bolt, Ft= Tension/N =	0.00	kN	
Actual Shear / Bolt, Fs= Shear/N =	11.25	kN	
Assumed Bolt Dia, d =	20	mm	
Gross Area of bolt, Ag = Л d^2 / 4 =	314.2	mm^2	
Tensile Stress Area of Bolt, At =	245.0	mm^2	
Tension strength of bolt, pt = (4.6 grade)	240.0	N / mm^2	
Shear strength of bolt, ps = (4.6 grade)	160.0	N / mm^2	
Tension Capacity of bolt, Pt = 0.8 pt At =	47.04	kN	Safe
Shear Capacity, Ps = ps. Ag =	50.27	kN	Safe
Combined Shear & Tension = Ft/Pt+Fs/Ps	0.22	<1.4 Safe	

APPENDIX C
BASE PLATE CALULATION DESIGN AISC

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

Base Plate Design (as per AISC) Sec. J-8

Input :

Axial Load (P)	=	90	kN				
Flange Width (wf)	=	203.6	mm				
Flange Thickness(tf)	=	11	mm				
Web Depth (dw)	=	203.2	mm				
Web Thickness (tw)	=	7.2	mm				
Base Plate Width (b)	=	250	mm	or	204	mm	
Base Plate Depth (d)	=	250	mm	or	245	mm	
Concrete Support Condition	=	0.35	User input				
Yield Strength of Steel (fy)	=	275	Mpa				
Compressive strength of concrete (f'c)	=	20	Mpa				

Check for Bearing :

Allowable Bearing Pressure (Fp_{All}) = 0.35*f'c = 7.00 Mpa
Allowable Base Plate Area (A_{all}) = (P*1000)/Fpall = 12,857 mm2
Actual Bearing Pressure (Fp_{act}) = (P*1000)/(b*d) = 1.44 Mpa
Bearing Pressure Ratio = Fpact/Fpall = 1.44/7.00 = 0.21 [Pass]

Check for Bending :

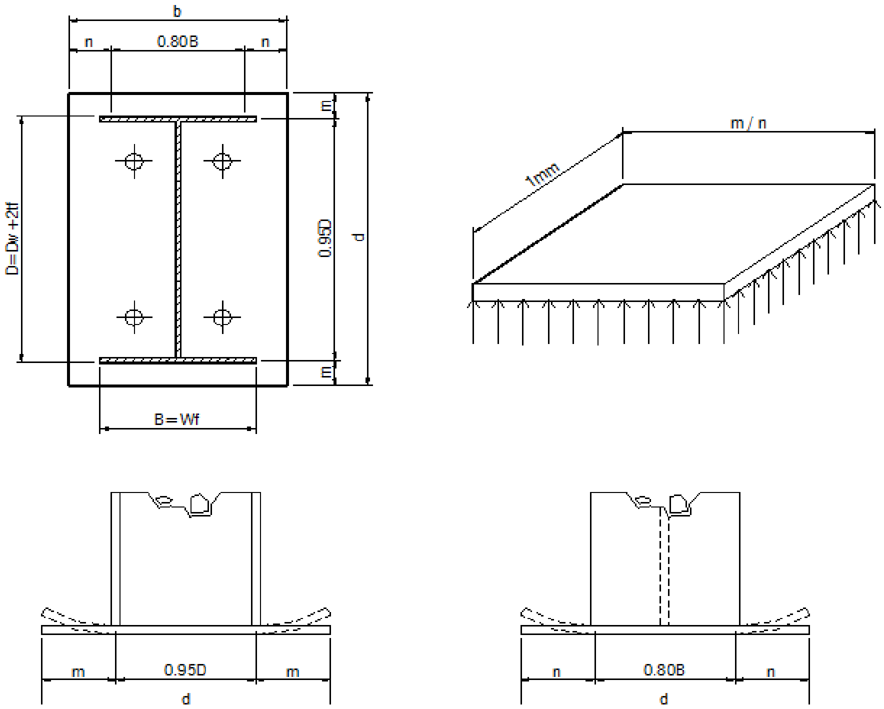
Constant (n) = [b-(0.8*wf)]/2 = 43.56 mm
Constant (m) = [d-(0.95*dw)]/2 = 28.48 mm
Constant (n') = Sqrt[(dw+2*tf)*wf]/4 = 58.83 mm
Constant (l) = Max (n, m & n') = 58.83 mm
Bending Moment (M) = (Fpact*l^2)/2 = 2,492 N.mm

Consider 1mm strip for the bending :

0.75*fy = (6*M)/(bt^2)
Base Plate Thicksness (t) = Sqrt[(6*M)/(1m*0.75*fy)] = 8.51 mm

Therefore :

Base Plate Size & thickness = **250 mm x 250 mm x 12 mm**



APPENDIX D
BASE PLATE CALULATION DESIGN EUROCODE

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

Basic design data:

Design axial load:	N_{Ed}=	-90.0 kN	↓
Design moment load:	M_{Ed}=	45.0 kNm	
Base plate steel grade: S275	f_{yp}=	275 N/mm²	*(EN 1993-1-1, Table 3.1)*
Foundation concrete class: C30/37	f_{ck}=	30.0 N/mm²	*(EN 1992-1-1, Table 3.1)*
Partial factors			
▪ concrete:	γ_c=	1.50	*(EN 1992-1-1, Table 2.1N)*
▪ steel:	γ_{M0}=	1.00	*(EN 1993-1-1, § 6.1 (1))*
▪ anchor bolts:	γ_{Mb}=	1.25	*(EN 1993-1-8, Table 2.1)*
Long-term effect coefficients			
▪ compressive strength:	α_{cc}=	0.85	*(EN 1992-1-1, § 3.1.6 (1))*
▪ tensile strength:	α_{ct}=	1.00	*(EN 1992-1-1, § 3.1.6 (2))*
Column section 203X203X46			
▪ height:	h_c=	203 mm	
▪ width:	b_{fc}=	204 mm	
▪ web thickness:	t_{wc}=	7.2 mm	
▪ flange thickness:	t_{fc}=	11.0 mm	

Determination of base plate dimensions, based on compression force

N_{Ed}= -90.0 kN	γ_c= 1.50	α_{cc}= 0.85
M_{Ed}= 45.0 kNm	γ_{M0}= 1.00	

Column section type:	203X203X46		
h_c= 203 mm	b_{fc}= 204 mm	t_{wc}= 7.2 mm	t_{fc}= 11.0 mm

Foundation concrete class:	C40/50	f_{ck}= 40.0 N/mm²	*(EN 1992-1-1, Table 3.1)*
Presumed base plate thickness:	16 < t ≤ 40		
Base plate steel grade:	S275	f_{yp}= 265 N/mm²	*(EN 1993-1-1, Table 3.1)*
Foundation joint coefficient: β_j=	2/3		*(EN 1993-1-8, § 6.2.5 (7))**
	1.50		*concrete bearing strength enhancement ratio***
	279.1 kN		*maximum compressive force acting on the foundation*
$N_{j,Ed}$= 2 $_{max}$($F_{C,Ed}$)=	558.3 kN		*assumed axial compressive load*
f_{jd}= $\beta_j\alpha f_{cd}$=	22.67 N/mm²		*design bearing strength of the foundation joint*
	20935 mm²		*preliminary estimate of the base plate area*

A_{c0}= 20935 mm² < $0.95h_cb_{fc}$= 39303 mm² adopt a 'large projection' base plate

A	B	C
2	603.2	-9422.7

14.9 mm *additional bearing width calculation*

< 0

Check for overlapping T-stubs:

2c = 29.8 mm < h_c-2t_{fc}= 181.2 mm (check passed)

Required minimum plan dimensions and thickness of the base plate:

233.4 mm 233.0 mm ('large projection' base plate)

Determination of base plate thickness and anchor bolt dimensions, based on tension force					
N_{Ed}=	-90.0 kN	γ_c=	1.50	γ_{Mb}=	1.25
M_{Ed}=	45.0 kNm	γ_{M0}=	1.00	β_t=	1.00
h_c=	203 mm	t_{fc}=	11.0 mm		
			189.1 kN		*maximum tensile force acting on the foundation*
Base plate steel grade:			S275	f_{yp}= 265.0 N/mm²	*(EN 1993-1-1, Table 3.1)*
Foundation concrete class:			C40/50	f_{ck}= 40.0 N/mm²	*(EN 1992-1-1, Table 3.1)*
Number of bolt rows *n* :			2		
Estimated bond conditions:			'good'	η_1= 1.00	*(EN 1992-1-1, § 8.4.1 (2))*
Presumed bolt diameter ϕ :			M20	η_2= 1.00	*(EN 1992-1-1, § 8.4.1 (2))*
Stress area of the bolt:		A_s=	245 mm²		*(EN ISO 898-1)*
Bolt class:			8.8	f_{ub}= 800 N/mm²	*(EN ISO 898-1)*
Type of the bolt bar:			plain bar	k= 2.25	
Basic anchorage length:		L_b=	900 mm		
			3.68 N/mm²		*design bond strength of the concrete*
Anchor bolt size (first estimate):					
			82 mm²		*required bolt stress area (first estimate)*
		Required bolt diameter:	M16		*first estimate, based on the selected bolt class*
Anchor bolt resistance:					
		$F_{t,bond,Rd}=\pi\phi l_b f_{bd}$=	208.3 kN		*design bond anchorage resistance of the bolt*
			141.1 kN		*design tensile resistance of the bolt*
		$F_{t,anchor,Rd}=\min(F_{t,bond,Rd};F_{t,Rd})$=	141.1 kN		*design resistance of a single anchor bolt*
Anchor bolt check:					
		$2nF_{t,anchor,Rd}$=	564.5 kN		
		$_{max}F_{T,Ed}$=	189.1 kN		
Required minimum base plate thickness:					
			7.5 mm		

APPENDIX E
DETAILED REPORT OF THE SOFTWARE ANALYSIS FOR BS 5950

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

STAAD INPUT:-

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 11-Apr-18

JOB NAME BS 5950-STEEL FABRICATION UNIT

JOB CLIENT STRUCTURAL DESIGN

ENGINEER NAME SR

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 18; 2 0 7.7 18; 3 12 0 18; 4 24 0 18; 5 36 0 18; 6 12 8.33 18;

7 24 8.33 18; 8 36 7.7 18; 9 18 8.645 18; 14 0 0 24; 16 36 0 24;

20 18 8.645 24; 23 0 0 48; 24 0 7.7 48; 25 12 0 48; 26 24 0 48; 27 36 0 48;

28 12 8.33 48; 29 24 8.33 48; 30 36 7.7 48; 31 18 8.645 48; 54 0 7.7 24;

58 36 7.7 24; 62 0 0 30; 63 0 7.7 30; 64 36 0 30; 67 36 7.7 30; 68 18 8.645 30;

71 0 0 36; 72 0 7.7 36; 73 36 0 36; 76 36 7.7 36; 77 18 8.645 36; 80 0 0 42;

81 0 7.7 42; 82 36 0 42; 85 36 7.7 42; 86 18 8.645 42; 130 9.0041 8.1727 18;

132 9.0041 8.1727 48; 135 9.0041 8.1727 24; 136 9.0041 8.1727 30;

137 9.0041 8.1727 36; 138 9.0041 8.1727 42; 139 26.9959 8.1727 18;

141 26.9959 8.1727 42; 142 26.9959 8.1727 48; 143 26.9959 8.1727 36;

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MEMBER INCIDENCES

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POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
TYPE STEEL
STRENGTH FY 253200 FU 407800 RY 1.5 RT 1.2
END DEFINE MATERIAL
MEMBER PROPERTY BRITISH
1 TO 4 13 14 21 TO 24 56 57 64 65 72 73 158 175 233 238 TO 240 259 260 419 -
420 TO 426 TABLE ST UB762X267X134
MEMBER PROPERTY BRITISH
5 TO 8 15 16 19 20 25 TO 28 58 59 62 63 66 67 70 71 74 75 78 79 182 185 199 -
203 TABLE ST UB762X267X134
80 84 TO 88 92 TO 95 183 192 TO 195 201 204 206 208 210 216 TO 223 -
403 TO 418 427 TO 446 TABLE ST PIP1688.0
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368 370 372 374 376 378 380 382 384 386 388 390 392 -
394 TABLE ST UB406X178X54
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345 347 349 351 353 355 357 359 361 363 365 367 369 371 373 375 377 379 381 -
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CONSTANTS
MATERIAL STEEL ALL
SUPPORTS
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MEMBER RELEASE
183 192 TO 195 201 204 206 208 210 330 332 334 336 338 340 342 344 346 348 -
350 352 354 356 358 360 362 364 366 368 370 372 374 376 378 380 382 384 386 -
388 390 392 394 START MY MZ
183 192 TO 195 201 204 206 208 210 330 332 334 336 338 340 342 344 346 348 -
350 352 354 356 358 360 362 364 366 368 370 372 374 376 378 380 382 384 386 -
388 390 392 394 END MY MZ
LOAD 1 LOADTYPE Dead  TITLE DEAD LOAD
SELFWEIGHT Y -1
LOAD 2 LOADTYPE Dead  TITLE SUPER DEAD LOAD
FLOOR LOAD
YRANGE 0 4 FLOAD -0.1 GY
MEMBER LOAD
15 16 19 20 58 59 62 63 66 67 70 71 74 75 78 79 UNI GY -0.9
5 TO 8 25 TO 28 182 185 199 203 UNI GY -0.45
LOAD 3 LOADTYPE Live  TITLE LIVE LOAD
FLOOR LOAD
YRANGE 0 4 FLOAD -4 GY
LOAD COMB 4 1.4DL+1.4SDL+1.6LL
1 1.4 2 1.4 3 1.6
LOAD COMB 5 1.2DL+1.2SDL+1.2LL
1 1.2 2 1.2 3 1.2
PERFORM ANALYSIS PRINT ALL
PARAMETER 1
CODE BS5950
STEEL MEMBER TAKE OFF ALL
PARAMETER 2
```

CODE BS5950

CHECK CODE ALL

FINISH

STAAD OUTPUT:

Failed Members

There is no data of this type.

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
Max Fx	23	26	4:1.4DL+1.4SI	**340.832**	-1.652	-0.820	0.000	0.000	-0.000
Min Fx	251	161	4:1.4DL+1.4SI	**-109.618**	130.264	0.017	0.000	0.018	280.785
Max Fy	241	154	4:1.4DL+1.4SI	-19.211	**133.446**	-0.008	-0.001	0.001	289.932
Min Fy	328	153	4:1.4DL+1.4SI	-19.211	**-133.446**	0.008	0.001	0.001	289.932
Max Fz	419	254	4:1.4DL+1.4SI	35.003	-2.875	**2.255**	-0.010	-3.298	12.227
Min Fz	240	155	4:1.4DL+1.4SI	29.218	48.266	**-2.183**	-0.000	3.766	135.420
Max Mx	221	145	4:1.4DL+1.4SI	0.412	2.056	-0.034	**1.676**	0.160	-0.086
Min Mx	217	135	4:1.4DL+1.4SI	0.412	2.056	0.034	**-1.676**	-0.160	-0.086
Max My	419	2	4:1.4DL+1.4SI	27.923	-2.875	2.255	-0.010	**5.385**	23.295
Min My	233	24	4:1.4DL+1.4SI	22.413	-48.266	2.183	0.000	**-4.311**	-43.163
Max Mz	425	72	4:1.4DL+1.4SI	85.119	-45.484	-1.795	-0.002	-3.884	**349.667**
Min Mz	426	76	4:1.4DL+1.4SI	85.119	45.484	-1.795	0.002	-3.884	**-349.667**

Reaction Summary

	Node	L/C	Horizontal FX (kN)	Vertical FY (kN)	Horizontal FZ (kN)	Moment MX (kNm)	MY (kNm)	MZ (kNm)
Max FX	71	4:1.4DL+1.4SI	**45.576**	120.746	-16.954	0.000	0.000	0.000
Min FX	73	4:1.4DL+1.4SI	**-45.576**	120.746	-16.954	0.000	0.000	0.000
Max FY	26	4:1.4DL+1.4SI	1.652	**340.832**	-0.820	0.000	0.000	0.000
Min FY	73	3:LIVE LOAD	-0.047	**-0.297**	0.183	0.000	0.000	0.000
Max FZ	1	4:1.4DL+1.4SI	2.981	67.626	**15.156**	0.000	0.000	0.000
Min FZ	71	4:1.4DL+1.4SI	45.576	120.746	**-16.954**	0.000	0.000	0.000
Max MX	1	1:DEAD LOAD	1.689	39.135	7.572	**0.000**	0.000	0.000
Min MX	1	1:DEAD LOAD	1.689	39.135	7.572	**0.000**	0.000	0.000
Max MY	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	**0.000**	0.000
Min MY	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	**0.000**	0.000
Max MZ	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	**0.000**
Min MZ	1	1:DEAD LOAD	1.689	39.135	7.572	0.000	0.000	**0.000**

Node Displacement Summary

	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	261	4:1.4DL+1.4SI	**6.034**	-0.116	0.140	6.036	-0.000	0.000	-0.001
Min X	260	4:1.4DL+1.4SI	**-6.034**	-0.116	0.140	6.036	-0.000	-0.000	0.001
Max Y	86	3:LIVE LOAD	-0.000	**1.631**	0.123	1.635	0.000	-0.000	-0.000
Min Y	77	4:1.4DL+1.4SI	-0.000	**-73.748**	1.622	73.766	-0.004	0.000	-0.000
Max Z	20	4:1.4DL+1.4SI	-0.000	-60.519	**6.148**	60.830	0.005	0.000	-0.000
Min Z	86	4:1.4DL+1.4SI	-0.000	-42.715	**-2.942**	42.817	-0.002	-0.000	-0.000
Max rX	9	4:1.4DL+1.4SI	-0.000	-1.200	5.782	5.905	**0.014**	-0.000	0.000
Min rX	31	4:1.4DL+1.4SI	-0.000	-1.293	-2.129	2.490	**-0.012**	0.000	0.000
Max rY	257	4:1.4DL+1.4SI	4.823	-0.064	-0.135	4.826	-0.000	**0.001**	-0.001
Min rY	256	4:1.4DL+1.4SI	-4.823	-0.064	-0.135	4.826	-0.000	**-0.001**	0.001
Max rZ	143	4:1.4DL+1.4SI	1.369	-46.216	0.736	46.242	-0.004	0.000	**0.005**
Min rZ	137	4:1.4DL+1.4SI	-1.369	-46.216	0.736	46.242	-0.004	-0.000	**-0.005**
Max Rst	77	4:1.4DL+1.4SI	-0.000	-73.748	1.622	**73.766**	-0.004	0.000	-0.000

The following output of the design analysis is with respect to the BS 5950.

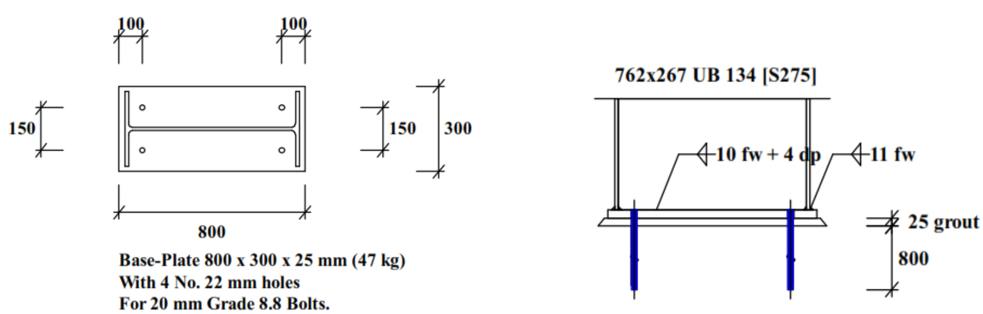

FIGURE 5.14: Geometric considerations for BS 5950

MEMBERS

Column

Column type	:	Prismatic member
Section	:	UB 762x267x134
Material	:	S275
Longitudinal offset	:	0 mm

DEMANDS

Description	Pu [KN]	Mu22 [KN*m]	Mu33 [KN*m]	Vu2 [KN]	Vu3 [KN]	Load type
CASE1	340.832	0.00	0.00	45.576	15.156	Design
CASE2	-0.297	0.00	0.00	-45.576	-16.954	Design

The most critical case have been used for the analysis of the steel column base plate.

Step 2a: Plate Compression Bending

e=L1	8.2	8.2 mm	
$M_{app}=p\cdot e^2/2$	$9.0\cdot 8.2^2/2$	303 Nmm/mm	
$tp=\sqrt{(6\cdot M_{app}/p_y)}$	$\sqrt{(6\cdot 303/265)}$	2.6 mm	OK
Note: Axial Load Axial	Using Elastic Modulus Zp (4.13.2.2)		

Step 4: Shear

Base Friction

Friction Fr=0.30•Fc	0.30••340.8 kN	102.2 kN	

Bolt Bearing

Bolt Shear Bs=Fn(ShrCap, tg)	91.9, 50	91.9 kN	
Concrete Bearing Cb=3•Ø²•0.4•Fcu	3•20²•0.4•15 (No Shear Reinf.)	7.2 kN	
Plate Bearing Pb=Fn(pb,e,Ø,t,kbs)	460, 100, 20, 25, 1.00	230.0 kN	
Bolt Bearing Bb=pb•Ø•tk	1000•20•25	500.0 kN	
Pss=Min(Bs, Cb, Pb, Bb)•nbs	Min(91.9, 7.2, 230.0, 500.0) = 7.2•2	14.4 kN	
Pts=Min(Bsten, Cb, Pb, Bb)•nbt	Min(91.9, 7.2, 230.0, 500.0) = 7.2•2, (no tension)	14.4 kN	

Total Shear Capacity

Total Cap=Fr+Pss+Pts	102.2 + 14.4 + 14.4	131.0 kN	OK

Step 5: Flange & Web Welds

Load dispersal	Flanges resist Moment and Axial, Web resists Axial and Shear. Direct Bearing therefore design for tensile forces only.
Areas A, Af, Aw	170.6, 2 x 41.0, 86.3 cm²

Flange Welds

Fapp=F•Af/A	340.8•41.0/170.6 No Resultant Tensile Force	0.0 kN	

Web Welds

Web weld load=Fv/(D-2(fw+T))	45.5/(750.0 - 2(11 +15.5))	0.07 kN/mm	
Fcap w=2•0.7•leg•Py	2•0.7•14•220	4.31 kN/mm	OK

APPENDIX F
DETAILED REPORT OF THE SOFTWARE ANALYSIS FOR AISC

NOTE: ALL THE ANALYSIS IN THIS DISSERTATION IS ONLY FOR RESEARCH WORK PURPOSE AND NOT FOR ANY PRACTICAL USE

STAAD INPUT:-

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 11-Apr-18

JOB NAME AISC-STEEL FABRICATION UNIT

JOB CLIENT STRUCTURAL DESIGN

ENGINEER NAME SR

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 18; 2 0 7.7 18; 3 12 0 18; 4 24 0 18; 5 36 0 18; 6 12 8.33 18;

7 24 8.33 18; 8 36 7.7 18; 9 18 8.645 18; 14 0 0 24; 16 36 0 24;

20 18 8.645 24; 23 0 0 48; 24 0 7.7 48; 25 12 0 48; 26 24 0 48; 27 36 0 48;

28 12 8.33 48; 29 24 8.33 48; 30 36 7.7 48; 31 18 8.645 48; 54 0 7.7 24;

58 36 7.7 24; 62 0 0 30; 63 0 7.7 30; 64 36 0 30; 67 36 7.7 30; 68 18 8.645 30;

71 0 0 36; 72 0 7.7 36; 73 36 0 36; 76 36 7.7 36; 77 18 8.645 36; 80 0 0 42;

81 0 7.7 42; 82 36 0 42; 85 36 7.7 42; 86 18 8.645 42; 130 9.0041 8.1727 18;

132 9.0041 8.1727 48; 135 9.0041 8.1727 24; 136 9.0041 8.1727 30;

137 9.0041 8.1727 36; 138 9.0041 8.1727 42; 139 26.9959 8.1727 18;

141 26.9959 8.1727 42; 142 26.9959 8.1727 48; 143 26.9959 8.1727 36;

144 26.9959 8.1727 30; 145 26.9959 8.1727 24; 152 0 4 48; 153 12 4 48;

154 24 4 48; 155 36 4 48; 160 0 4 42; 161 12 4 42; 162 24 4 42; 163 36 4 42;

164 12 0 42; 165 24 0 42; 188 25 4 48; 189 26 4 48; 190 27 4 48; 191 28 4 48;

192 29 4 48; 193 30 4 48; 194 31 4 48; 195 32 4 48; 196 33 4 48; 197 34 4 48;

198 35 4 48; 199 13 4 48; 200 14 4 48; 201 15 4 48; 202 16 4 48; 203 17 4 48;
204 18 4 48; 205 19 4 48; 206 20 4 48; 207 21 4 48; 208 22 4 48; 209 23 4 48;
210 1 4 48; 211 2 4 48; 212 3 4 48; 213 4 4 48; 214 5 4 48; 215 6 4 48;
216 7 4 48; 217 8 4 48; 218 9 4 48; 219 10 4 48; 220 11 4 48; 221 1 4 42;
222 2 4 42; 223 3 4 42; 224 4 4 42; 225 5 4 42; 226 6 4 42; 227 7 4 42;
228 8 4 42; 229 9 4 42; 230 10 4 42; 231 11 4 42; 232 13 4 42; 233 14 4 42;
234 15 4 42; 235 16 4 42; 236 17 4 42; 237 18 4 42; 238 19 4 42; 239 20 4 42;
240 21 4 42; 241 22 4 42; 242 23 4 42; 243 25 4 42; 244 26 4 42; 245 27 4 42;
246 28 4 42; 247 29 4 42; 248 30 4 42; 249 31 4 42; 250 32 4 42; 251 33 4 42;
252 34 4 42; 253 35 4 42; 254 0 3.85 18; 255 36 3.85 18; 256 0 3.85 24;
257 36 3.85 24; 258 0 3.85 30; 259 36 3.85 30; 260 0 3.85 36; 261 36 3.85 36;

MEMBER INCIDENCES

1 1 254; 2 3 6; 3 4 7; 4 5 255; 5 6 9; 6 7 9; 7 2 130; 8 8 139; 13 14 256;
14 16 257; 15 54 135; 16 58 145; 19 20 135; 20 20 145; 21 23 152; 22 25 153;
23 26 154; 24 27 155; 25 28 31; 26 29 31; 27 24 132; 28 30 142; 56 62 258;
57 64 259; 58 63 136; 59 67 144; 62 68 136; 63 68 144; 64 71 260; 65 73 261;
66 72 137; 67 76 143; 70 77 137; 71 77 143; 72 80 160; 73 82 163; 74 81 138;
75 85 141; 78 86 138; 79 86 141; 80 2 54; 84 54 63; 85 63 72; 86 72 81;
87 81 24; 88 8 58; 92 58 67; 93 67 76; 94 76 85; 95 85 30; 158 81 160;
175 85 163; 182 130 6; 183 130 135; 185 132 28; 192 138 132; 193 137 138;
194 136 137; 195 135 136; 199 139 7; 201 139 145; 203 142 29; 204 141 142;
206 143 141; 208 144 143; 210 145 144; 216 2 135; 217 135 9; 218 54 130;
219 130 20; 220 8 145; 221 145 9; 222 58 139; 223 139 20; 233 24 152;
238 153 28; 239 154 29; 240 155 30; 241 154 188; 242 153 199; 243 152 210;
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260 165 162; 261 161 153; 262 162 154; 296 188 189; 297 189 190; 298 190 191;
299 191 192; 300 192 193; 301 193 194; 302 194 195; 303 195 196; 304 196 197;
305 197 198; 306 198 155; 307 199 200; 308 200 201; 309 201 202; 310 202 203;
311 203 204; 312 204 205; 313 205 206; 314 206 207; 315 207 208; 316 208 209;
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323 215 216; 324 216 217; 325 217 218; 326 218 219; 327 219 220; 328 220 153;

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335 224 225; 336 213 224; 337 225 226; 338 214 225; 339 226 227; 340 215 226;
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347 230 231; 348 219 230; 349 231 161; 350 220 231; 351 232 233; 352 199 232;
353 233 234; 354 200 233; 355 234 235; 356 201 234; 357 235 236; 358 202 235;
359 236 237; 360 203 236; 361 237 238; 362 204 237; 363 238 239; 364 205 238;
365 239 240; 366 206 239; 367 240 241; 368 207 240; 369 241 242; 370 208 241;
371 242 162; 372 209 242; 373 243 244; 374 188 243; 375 244 245; 376 189 244;
377 245 246; 378 190 245; 379 246 247; 380 191 246; 381 247 248; 382 192 247;
383 248 249; 384 193 248; 385 249 250; 386 194 249; 387 250 251; 388 195 250;
389 251 252; 390 196 251; 391 252 253; 392 197 252; 393 253 163; 394 198 253;
403 81 132; 404 132 86; 405 24 138; 406 138 31; 407 85 142; 408 142 86;
409 30 141; 410 141 31; 411 63 137; 412 137 68; 413 72 136; 414 136 77;
415 67 143; 416 143 68; 417 76 144; 418 144 77; 419 254 2; 420 255 8;
421 256 54; 422 257 58; 423 258 63; 424 259 67; 425 260 72; 426 261 76;
427 2 256; 428 54 254; 429 254 14; 430 256 1; 431 254 256; 432 63 260;
433 72 258; 434 258 71; 435 260 62; 436 258 260; 437 67 261; 438 76 259;
439 259 73; 440 261 64; 441 259 261; 442 8 257; 443 58 255; 444 255 16;
445 257 5; 446 255 257;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
TYPE STEEL
STRENGTH FY 253200 FU 407800 RY 1.5 RT 1.2
END DEFINE MATERIAL
MEMBER PROPERTY BRITISH
1 TO 4 13 14 21 TO 24 56 57 64 65 72 73 158 175 233 238 TO 240 259 260 419 -
```

```
420 TO 426 TABLE ST UB686X254X125
MEMBER PROPERTY BRITISH
5 TO 8 15 16 19 20 25 TO 28 58 59 62 63 66 67 70 71 74 75 78 79 182 185 199 -
203 TABLE ST UB686X254X125
80 84 TO 88 92 TO 95 183 192 TO 195 201 204 206 208 210 216 TO 223 -
403 TO 418 427 TO 446 TABLE ST PIP1688.0
330 332 334 336 338 340 342 344 346 348 350 352 354 356 358 360 362 364 366 -
368 370 372 374 376 378 380 382 384 386 388 390 392 -
394 TABLE ST UB406X178X54
241 TO 243 250 TO 252 257 258 261 262 296 TO 329 331 333 335 337 339 341 343 -
345 347 349 351 353 355 357 359 361 363 365 367 369 371 373 375 377 379 381 -
383 385 387 389 391 393 TABLE ST UB610X229X113
CONSTANTS
MATERIAL STEEL ALL
SUPPORTS
1 3 TO 5 14 16 23 25 TO 27 62 64 71 73 80 82 164 165 PINNED
MEMBER RELEASE
183 192 TO 195 201 204 206 208 210 330 332 334 336 338 340 342 344 346 348 -
350 352 354 356 358 360 362 364 366 368 370 372 374 376 378 380 382 384 386 -
388 390 392 394 START MY MZ
183 192 TO 195 201 204 206 208 210 330 332 334 336 338 340 342 344 346 348 -
350 352 354 356 358 360 362 364 366 368 370 372 374 376 378 380 382 384 386 -
388 390 392 394 END MY MZ
LOAD 1 LOADTYPE Dead  TITLE DEAD LOAD
SELFWEIGHT Y -1
LOAD 2 LOADTYPE Dead  TITLE SUPER DEAD LOAD
FLOOR LOAD
YRANGE 0 4 FLOAD -0.1 GY
MEMBER LOAD
15 16 19 20 58 59 62 63 66 67 70 71 74 75 78 79 UNI GY -0.9
5 TO 8 25 TO 28 182 185 199 203 UNI GY -0.45
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LOAD 3 LOADTYPE Live TITLE LIVE LOAD

FLOOR LOAD

YRANGE 0 4 FLOAD -3.59 GY

LOAD COMB 4 1.4DL+1.4SDL

1 1.4 2 1.4

LOAD COMB 5 1.2DL+1.2SDL+1.6LL

1 1.2 2 1.2 3 1.6

LOAD COMB 6 1.2DL+1.2SDL+LL

1 1.2 2 1.2 3 1.0

LOAD COMB 7 1.2DL+1.2SDL

1 1.2 2 1.2

LOAD COMB 8 0.9DL+0.9SDL

1 0.9 2 0.9

PERFORM ANALYSIS PRINT ALL

PARAMETER 1

CODE LRFD

STEEL MEMBER TAKE OFF ALL

PARAMETER 2

CODE LRFD

CHECK CODE ALL

FINISH

STAAD OUTPUT:

Failed Members

There is no data of this type

Reaction Summary

	Node	L/C	Horizontal	Vertical	Horizontal	Moment		
			FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
Max FX	71	4:1.4DL+1.4SD	**44.013**	121.866	-19.237	0.000	0.000	0.000
Min FX	73	4:1.4DL+1.4SD	**-44.013**	121.866	-19.237	0.000	0.000	0.000
Max FY	25	5:1.2DL+1.2SD	-1.847	**300.395**	-0.680	0.000	0.000	0.000
Min FY	73	3:LIVE LOAD	-0.050	**-0.302**	0.189	0.000	0.000	0.000
Max FZ	1	4:1.4DL+1.4SD	2.814	69.887	**17.025**	0.000	0.000	0.000
Min FZ	71	4:1.4DL+1.4SD	44.013	121.866	**-19.237**	0.000	0.000	0.000
Max MX	1	1:DEAD LOAD	1.579	39.380	8.272	**0.000**	0.000	0.000
Min MX	1	1:DEAD LOAD	1.579	39.380	8.272	**0.000**	0.000	0.000
Max MY	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	**0.000**	0.000
Min MY	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	**0.000**	0.000
Max MZ	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	**0.000**
Min MZ	1	1:DEAD LOAD	1.579	39.380	8.272	0.000	0.000	**0.000**

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

	Beam	Node	L/C	Axial	Shear		Torsion	Bending	
				Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
Max Fx	22	25	5:1.2DL+1.2SD	**300.395**	1.847	-0.680	0.000	-0.000	0.000
Min Fx	182	6	4:1.4DL+1.4SD	**-118.857**	-21.570	0.065	0.343	0.234	45.854
Max Fy	241	154	5:1.2DL+1.2SD	-17.572	**119.390**	-0.010	-0.001	0.023	259.974
Min Fy	328	153	5:1.2DL+1.2SD	-17.572	**-119.390**	0.010	0.001	0.023	259.974
Max Fz	420	255	4:1.4DL+1.4SD	35.097	2.692	**2.296**	0.012	-3.355	-11.710
Min Fz	240	155	4:1.4DL+1.4SD	25.385	18.965	**-1.929**	0.000	2.783	36.178
Max Mx	183	130	4:1.4DL+1.4SD	36.877	1.300	-0.000	**1.831**	0.000	0.000
Min Mx	201	139	4:1.4DL+1.4SD	36.877	1.300	0.000	**-1.831**	0.000	0.000
Max My	420	8	4:1.4DL+1.4SD	28.514	2.692	2.296	0.012	**5.487**	-22.074
Min My	240	30	4:1.4DL+1.4SD	19.058	18.965	-1.929	0.000	**-4.353**	-33.992
Max Mz	425	72	4:1.4DL+1.4SD	85.071	-43.902	-1.856	-0.003	-4.034	**337.343**
Min Mz	426	76	4:1.4DL+1.4SD	85.071	43.902	-1.856	0.003	-4.034	**-337.343**

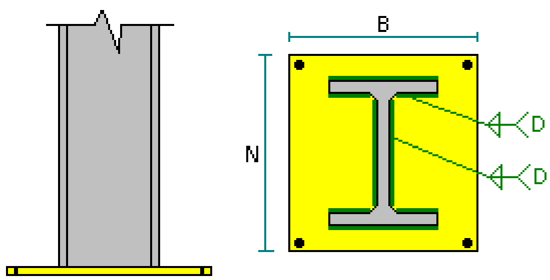
Node Displacement Summary

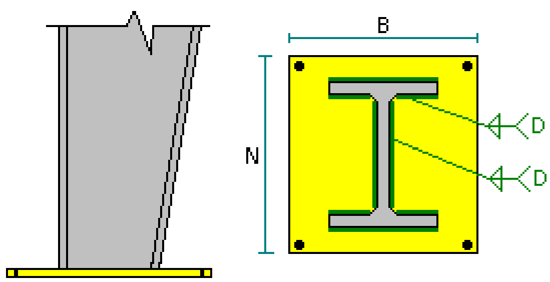
	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	261	4:1.4DL+1.4SD	**7.458**	-0.124	0.160	7.461	-0.000	0.000	-0.001
Min X	260	4:1.4DL+1.4SD	**-7.458**	-0.124	0.160	7.461	-0.000	-0.000	0.001
Max Y	86	3:LIVE LOAD	-0.000	**1.824**	0.149	1.830	0.000	-0.000	-0.000
Min Y	77	4:1.4DL+1.4SD	-0.000	**-90.720**	1.852	90.739	-0.004	0.000	-0.000
Max Z	20	4:1.4DL+1.4SD	-0.000	-71.319	**7.153**	71.677	0.007	0.000	-0.000
Min Z	86	4:1.4DL+1.4SD	-0.000	-52.703	**-3.586**	52.825	-0.003	-0.000	-0.000
Max rX	9	4:1.4DL+1.4SD	-0.000	-1.586	6.743	6.927	**0.016**	-0.000	0.000
Min rX	31	4:1.4DL+1.4SD	-0.000	-1.386	-2.885	3.201	**-0.013**	0.000	0.000
Max rY	257	4:1.4DL+1.4SD	5.737	-0.060	-0.160	5.740	-0.000	**0.001**	-0.001
Min rY	256	4:1.4DL+1.4SD	-5.737	-0.060	-0.160	5.740	-0.000	**-0.001**	0.001
Max rZ	143	4:1.4DL+1.4SD	1.701	-56.822	0.813	56.853	-0.004	0.000	**0.007**
Min rZ	137	4:1.4DL+1.4SD	-1.701	-56.822	0.813	56.853	-0.004	-0.000	**-0.007**
Max Rst	77	4:1.4DL+1.4SD	-0.000	-90.720	1.852	**90.739**	-0.004	0.000	-0.000

BASE PLATE DESIGN ANALYSIS FROM RAM ELEMENTS:

GENERAL INFORMATION

Connector





MEMBERS

Column

Column type	:	Prismatic member
Section	:	UB 762x267x197
Material	:	S275
Longitudinal offset	:	0 mm

CONNECTOR

Base plate

Connection type	:	Unstiffened
Position on the support	:	Center
N: Longitudinal dimension	:	800 mm
B: Transversal dimension	:	300 mm
Thickness	:	24.7 mm
Material	:	S275
Column weld	:	E70XX
Outer welds flanges only	:	No
D: Column weld size (1/16 in)	:	5
Override A2/A1 ratio	:	No
Include shear lug	:	No

Support

With pedestal	:	No
Longitudinal dimension	:	1250 mm
Transversal dimension	:	900 mm
Thickness	:	1000 mm
Material	:	C50-60
Include grouting	:	Yes
Grout thickness	:	25 mm

Anchor

Anchor position	:	Longitudinal position
Rows number per side	:	1
Anchors per row	:	2
Longitudinal edge distance on the plate	:	100 mm
Transverse edge distance on the plate	:	75 mm
Anchor type	:	Headed
Head type	:	Square
Include lock nut	:	No
Anchor	:	M_24
Effective embedment depth	:	800 mm
Total length	:	881.38 mm
Material	:	A36
Fy	:	0.248 kN/mm2
Fu	:	0.4 kN/mm2
Cracked concrete	:	No
Brittle steel	:	No
Anchors welded to base plate	:	No
Anchor reinforcement		
Type of reinforcement	:	Primary
Tension reinforcement	:	No
Shear reinforcement	:	No

Design code: AISC 360-10 LRFD, ACI 318-08

DEMANDS

Description	Pu [KN]	Mu22 [KN*m]	Mu33 [KN*m]	Vu2 [KN]	Vu3 [KN]	Load type
CASE1	300.40	0.00	0.00	44.01	17.03	Design
CASE2	-0.30	0.00	0.00	-44.01	-19.24	Design

Design for major axis

Base plate (AISC 360-10 LRFD)

GEOMETRIC CONSIDERATIONS

Dimensions References	Unit	Value	Min. value	Max. value	Sta.	
Base plate						
Distance from anchor to edge	[mm]	63.00	6.35	--	✔	
Weld size	[1/16in]	5	4	--	✔	table J2.4

$w_{min} = w_{min}$

= **0.00635** table J2.4

DESIGN CHECK

Verification References	Unit	Capacity	Demand	Ctrl EQ	Ratio	
Concrete base						
Axial bearing	[KN/mm2]	0.04	0.00	CASE2	0.00	DG1 3.1.1;

$A_2 = ((B/N)*N_{cs})*N_{cs}$

= ((300[mm]/800[mm])*1250[mm])*1250[mm]

= **5.86E+05**[mm2] DG1 Sec 3.1.1

$A_1 = B*N$

= 300[mm]*800[mm]

= **2.40E+05**[mm2] DG1 Sec 3.1.1

$f_{p,\,max} = \phi*\min(0.85*f'_c*(A_2/A_1)^{1/2}, 1.7*f'_c)$

= 0.65*min(0.85*50[N/mm2]*(2.44)^1/2, 1.7*50[N/mm2])

= **43.16**[N/mm2] DG1 3.1.1

Base plate

Flexural yielding (bearing interface) [KN*m/m] 37.75 0.00 CASE2 **0.00** DG1 Sec 3.1.2

$\phi M_n = \phi^* F_y^* t_p^2/4$
= 0.9*275[N/mm2]*24.7[mm]^2/4
= **37.75**[kN*m/m] DG1 Eq. 3.3.13

$A_2 = ((B/N)^*N_{cs})^*N_{cs}$
= ((300[mm]/800[mm])*1250[mm])*1250[mm]
= **5.86E+05**[mm2] DG1 Sec 3.1.1

$A_1 = B^*N$
= 300[mm]*800[mm]
= **2.40E+05**[mm2] DG1 Sec 3.1.1

$m = (N - 0.95^*d_c)/2$
= (800[mm] - 0.95*769.8[mm])/2
= **34.35**[mm] DG1 Sec 3.1.2

$n = (B - 0.8^*b_c)/2$
= (300[mm] - 0.8*268[mm])/2
= **42.8**[mm] DG1 Sec 3.1.2

$P_p = \min(0.85^*f'_c{}^*A_1{}^*(A_2/A_1)^{1/2}, 1.7^*f'_c{}^*A_1)$
= min(0.85*50[N/mm2]*2.40E+05[mm2]*(2.44)$^{1/2}$, 1.7*50[N/mm2]*2.40E+05[mm2])
= **15937.5**[kN] Eq. J8-2

$X = (4^*d_c{}^*b_c/(d_c + b_c)^2)^*P/(\phi^*P_p)$
= (4*769.8[mm]*268[mm]/(769.8[mm] + 268[mm])^2)*0.302[kN]/(0.65*15937.5[kN])
= **0.000022** DG1 Sec 3.1.2

$\lambda = \min(2^*(X)^{1/2}/(1 + (1 - X)^{1/2}), 1.0)$
= min(2*(0.000022)$^{1/2}$/(1 + (1 - 0.000022)$^{1/2}$), 1.0)
= **0.004726** DG1 Sec 3.1.2

$n' = \lambda^*(d_c{}^*b_c)^{1/2}/4$
= 0.004726*(769.8[mm]*268[mm])$^{1/2}$/4
= **0.537**[mm] DG1 Sec 3.1.2

$l = \max(m, n, n')$
= max(34.35[mm], 42.8[mm], 0.537[mm])
= **42.8**[mm] DG1 Sec 3.1.2

$f_p = P/(B^*N)$
= 0.302[kN]/(300[mm]*800[mm])
= **0.001258**[N/mm2] DG1 Sec 3.1.2

$M_{pl} = f_p{}^*(l^2/2)$
= 0.001258[N/mm2]*(42.8[mm]^2/2)

= **0.001153**[kN*m/m] DG1 Sec 3.1.2

Flexural yielding (tension interface)	[KN*m/m]	37.75	37.55	CASE1	**0.99**	DG1 Eq. 3.3.13

$\phi M_n = \phi * F_y * t_p^2/4$
= 0.9*275[N/mm2]*24.7[mm]^2/4
= **37.75**[kN*m/m] DG1 Eq. 3.3.13

$M_{pT} = M_{strip}/B_{eff}$
= 4.47[kN*m]/119[mm]
= **37.55**[kN*m/m]

<u>Column</u>

Weld capacity	[KN/m]	1828.47	631.08	CASE1	**0.35**	p. 8-9, Sec. J2.5, Sec. J2.4, DG1 p. 35

LoadAngleFactor = 1 + 0.5*(sin(θ))$^{1.5}$
= 1 + 0.5*(sin(1.57))$^{1.5}$
= **1.5** p. 8-9

$F_w = 0.6 * F_{EXX}$*LoadAngleFactor
= 0.6*482.63[N/mm2]*1.5
= **434.37**[N/mm2] Sec. J2.5

$A_w = (2)^{1/2}/2 * D/16$ [in]*L
= (2)$^{1/2}$/2*5/16 [in]*1000[mm]
= **5612.66**[mm2] Sec. J2.4

$\phi R_w = \phi * F_w * A_w/L$
= 0.75*434.37[N/mm2]*5612.66[mm2]/1000[mm]
= **1.83**[kN/mm]

$b_{eff} = 2*L$
= 2*59.5[mm]
= **119**[mm] DG1 p. 35

Maximum weld load = T/b_{eff}
= 75.1[kN]/119[mm]
= **0.631**[kN/mm]

Elastic method weld shear capacity	[KN/m]	1218.98	32.08	CASE1	**0.03**	p. 8-9, Sec. J2.5, Sec. J2.4

LoadAngleFactor = 1 + 0.5*(sin(θ))$^{1.5}$
= 1 + 0.5*(sin(0))$^{1.5}$
= **1** p. 8-9

$F_w = 0.6 * F_{EXX}$*LoadAngleFactor
= 0.6*482.63[N/mm2]*1
= **289.58**[N/mm2] Sec. J2.5

$A_w = (2)^{1/2}/2 * D/16$ [in]*L
= (2)$^{1/2}$/2*5/16 [in]*1000[mm]
= **5612.66**[mm2] Sec. J2.4

$\phi R_w = \phi * F_w * A_w/L$
= 0.75*289.58[N/mm2]*5612.66[mm2]/1000[mm]
= **1.22**[kN/mm]

$f_v = V/L_{shear}$
$= 44.01$[kN]/1372[mm]
$=$ **0.0321**[kN/mm]

Elastic method weld axial capacity	[KN/m]	1828.47	308.16	CASE1	0.17	p. 8-9, Sec. J2.5, Sec. J2.4

$LoadAngleFactor = 1 + 0.5*(\sin(\theta))^{1.5}$
$= 1 + 0.5*(\sin(1.57))^{1.5}$
$=$ **1.5** — p. 8-9

$F_w = 0.6*F_{EXX}*LoadAngleFactor$
$= 0.6*482.63$[N/mm2]$*1.5$
$=$ **434.37**[N/mm2] — Sec. J2.5

$A_w = (2)^{1/2}/2*D/16$ [in]$*L$
$= (2)^{1/2}/2*5/16$ [in]$*1000$[mm]
$=$ **5612.66**[mm2] — Sec. J2.4

$\phi R_w = \phi*F_w*A_w/L$
$= 0.75*434.37$[N/mm2]$*5612.66$[mm2]$/1000$[mm]
$=$ **1.83**[kN/mm]

$f_a = P/L$
$= 300.4$[kN]$/974.8$[mm]
$=$ **0.308**[kN/mm]

$f_b = M*c/I$
$= 0$[kN*m]$*384.9$[mm]$/1.90E+08$[mm3]
$=$ **0**[kN/mm]

$f = f_b + f_a$
$= 0$[kN/mm] $+ 0.308$[kN/mm]
$=$ **0.308**[kN/mm]

Ratio **0.99**

Major axis
Anchors
GEOMETRIC CONSIDERATIONS

Dimensions References	Unit	Value	Min. value	Max. value	Sta.	
Anchors						
Anchor spacing	[mm]	150.00	96.00	--	✔	Sec. D.8.1

$s_{min} = 4*d_a$
$= 4*24$[mm]
$=$ **96**[mm] — Sec. D.8.1

Concrete cover	[mm]	313.00	76.20	--	✔	Sec. 7.7.1

IsConcreteCastAgainstEarth $\rightarrow$ **True**

Cover = 3 [in] — Sec. 7.7.1

Effective length	[mm]	815.60	--	984.40	✔	

DESIGN CHECK

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[KN]	102.96	75.10	CASE1	**0.73**	Eq. D-3

$A_{se} = \pi/4.0*(d_a - 0.9743\ [in]/n_t)^2$
$= \pi/4.0*(24[mm] - 0.9743\ [in]/8)^2$
$= \mathbf{343.29}[mm2]$ Sec.
D.5.1.1,
D.6.1.2

$f_{uta} = min(f_{uta}, 1.9*f_{ya}, 125\ [ksi])$
$= min(399.89[N/mm2], 1.9*248.21[N/mm2], 125\ [ksi])$
$= \mathbf{399.89}[N/mm2]$ Sec.
D.5.1.2

$\phi N_{sa} = \phi*n*A_{se,N}*f_{uta}$
$= 0.75*1*343.29[mm2]*399.89[N/mm2]$
$= \mathbf{102.96}[kN]$ Eq. D-3

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Breakout of anchor in tension	[KN]	285.80	75.10	CASE1	**0.26**	Eq. D-4, Sec. D.4.1.1

$c_{a1Left}<1.5*h_{ef} \rightarrow 375[mm]<1.5*800[mm] \rightarrow$ **True**

$c_{a1Left} = c_{a1Left}$
$= \mathbf{375}[mm]$ Sec.
D.5.2.1

$c_{a1Right}<1.5*h_{ef} \rightarrow 525[mm]<1.5*800[mm] \rightarrow$ **True**

$c_{a1Right} = c_{a1Right}$
$= \mathbf{525}[mm]$ Sec.
D.5.2.1

$c_{a2Top}<1.5*h_{ef} \rightarrow 925[mm]<1.5*800[mm] \rightarrow$ **True**

$c_{a2Top} = c_{a2Top}$
$= \mathbf{925}[mm]$ Sec.
D.5.2.1

$c_{a2Bot}<1.5*h_{ef} \rightarrow 325[mm]<1.5*800[mm] \rightarrow$ **True**

$c_{a2Bot} = c_{a2Bot}$
$= \mathbf{325}[mm]$ Sec.
D.5.2.1

IsCloseToThreeEdges $\rightarrow$ **True**

$h_{ef} = c_{amax}/1.5$
$= 925[mm]/1.5$
$= \mathbf{616.67}[mm]$ Sec.
D.5.2.3

$c_{a1Left}<1.5*h_{ef} \rightarrow 375[mm]<1.5*616.67[mm] \rightarrow$ **True**

$c_{a1Left} = c_{a1Left}$
$= \mathbf{375}[mm]$ Sec.
D.5.2.1

$c_{a1Right}<1.5*h_{ef} \rightarrow 525[mm]<1.5*616.67[mm] \rightarrow$ **True**

$c_{a1Right} = c_{a1Right}$
= **525**[mm] Sec.
D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*616.67[mm] → **False**

c_{a2Top} = 1.5*h_{ef}
= 1.5*616.67[mm]
= **925**[mm] Sec.
D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

$c_{a2Bot} = c_{a2Bot}$
= **325**[mm] Sec.
D.5.2.1

A_{Nc} = (c_{a1Left} + $c_{a1Right}$)*(c_{a2Top} + c_{a2Bot})
= (375[mm] + 525[mm])*(925[mm] + 325[mm])
= **1.13E+06**[mm2] Sec.
RD.5.2.1

A_{Nco} = 9*h_{ef}^2
= 9*616.67[mm]^2
= **3.42E+06**[mm2] Eq. D-6

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*616.67[mm])
= **0.805** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec.
D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec.
D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec.
D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (616.67[mm]> = 11 [in]) and (616.67[mm]<
= 25 [in]) → **True**

N_b = 16*λ*(f_c/(1 [psi]))$^{1/2}$*(h_{ef}/(1 [in]))$^{(5/3)}$ [lb]
= 16*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(616.67[mm]/(1 [in]))$^{(5/3)}$ [lb]
= **1233.75**[kN] Eq. D-8

N_{cb} = (A_{Nc}/A_{Nco})*$\psi_{ed,N}$*$\psi_{c,N}$*$\psi_{cp,N}$*N_b
= (1.13E+06[mm2]/3.42E+06[mm2])*0.805*1.25*1*1233.75[kN]
= **408.28**[kN] Eq. D-4

HighSeismicDesignCategory → **False**

$\phi N_{cb} = \phi * N_{cb}$
= 0.7*408.28[kN]
= **285.8**[kN] Sec.
D.4.1.1

Breakout of group of anchors in tension	[KN]	471.62	300.40	CASE1	**0.64**	Eq. D-5, Sec.

D.4.1.1
$A_{Nco} = 9 * h_{ef}^2$
= 9*250[mm]^2
= **5.63E+05**[mm2] Eq. D-6

$A_{Nc} = \min(A_{Nc}, n * A_{Nco})$
= min(1.13E+06[mm2], 4*5.63E+05[mm2])
= **1.13E+06**[mm2] Sec.
D.5.2.1

$\psi_{ec,Ny} = \min(1/(1 + 2*e'_N/(3*h_{ef})), 1)$
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,Nx} = \min(1/(1 + 2*e'_N/(3*h_{ef})), 1)$
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,N} = \psi_{ec,Nx} * \psi_{ec,Ny}$
= 1*1
= **1** Eq. D-9

$c_{a,min} < 1.5 * h_{ef} \rightarrow$ 325[mm]<1.5*250[mm] $\rightarrow$ **True**

$\psi_{ed,N} = 0.7 + 0.3 * c_{a,min}/(1.5 * h_{ef})$
= 0.7 + 0.3*325[mm]/(1.5*250[mm])
= **0.96** Eq. D-11

CrackedConcrete $\rightarrow$ **False**

$\psi_{c,N}$ = 1.25 Sec.
D.5.2.6

IsCastInPlaceAnchor $\rightarrow$ **True**

$\psi_{cp,N}$ = 1 Sec.
D.5.2.7

IsCastInPlaceAnchor $\rightarrow$ **True**

k_c = 24 Sec.
D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) $\rightarrow$ (True) and (True) and (250[mm]> = 11 [in]) and (250[mm]<
= 25 [in]) $\rightarrow$ **False**

$N_b = k_c * \lambda * (f_c/(1\ [psi]))^{1/2} * (h_{ef}/(1\ [in]))^{1.5}$ [lb]
= 24*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(250[mm]/(1 [in]))$^{1.5}$ [lb]
= **280.73**[kN] Eq. D-7

$N_{cbg} = (A_{Nc}/A_{Nco}) * \psi_{ec,N} * \psi_{ed,N} * \psi_{c,N} * \psi_{cp,N} * N_b$

= (1.13E+06[mm2]/5.63E+05[mm2])*1*0.96*1.25*1*280.73[kN]
= **673.74**[kN] Eq. D-5

HighSeismicDesignCategory → **False**

$\phi N_{cbg} = \phi * N_{cbg}$
= 0.7*673.74[kN]
= **471.62**[kN] Sec. D.4.1.1

Pullout of anchor in tension	[KN]	330.85	75.10	CASE1	**0.23**	Sec. D.4.1.1

$A_{brg} = F^2 - A_g$
= 36[mm]^2 - 452[mm2]
= **844**[mm2]

IsHeadedBolt → **True**

$N_p = 8 * A_{brg} * f_c$
= 8*844[mm2]*50[N/mm2]
= **337.6**[kN] Eq. D-15

CrackedConcrete → **False**

$\psi_{c,P} = 1.4$ Sec. D.5.3.6

$N_{pn} = \psi_{c,P} * N_p$
= 1.4*337.6[kN]
= **472.64**[kN] Eq. D-14

HighSeismicDesignCategory → **False**

$\phi N_{pn} = \phi * N_{pn}$
= 0.7*472.64[kN]
= **330.85**[kN] Sec. D.4.1.1

Anchor shear	[KN]	42.83	11.00	CASE1	**0.26**	Eq. D-20, Sec. D.6.1.3

$A_{se} = \pi/4.0*(d_a - 0.9743\ [in]/n_t)^2$
= π/4.0*(24[mm] - 0.9743 [in]/8)^2
= **343.29**[mm2] Sec. D.5.1.1, D.6.1.2

$f_{uta} = \min(f_{uta}, 1.9*f_{ya}, 125\ [ksi])$
= min(399.89[N/mm2], 1.9*248.21[N/mm2], 125 [ksi])
= **399.89**[N/mm2] Sec. D.5.1.2

HasGroutPad → **True**

$\phi V_{sa} = 0.8*\phi*0.6*n*A_{se,V}*f_{uta}$
= 0.8*0.65*0.6*1*343.29[mm2]*399.89[N/mm2]
= **42.83**[kN] Eq. D-20, Sec. D.6.1.3

Breakout of anchor in shear [KN] 144.28 11.00 CASE1 **0.08** Sec. D.4.1.1

c_{a2Left}<1.5*c_{a1} → 375[mm]<1.5*325[mm] → **True**

c_{a2Left} = c_{a2Left}
= **375**[mm] Sec. D.6.2.1

$c_{a2Right}$<1.5*c_{a1} → 525[mm]<1.5*325[mm] → **False**

$c_{a2Right}$ = 1.5*c_{a1}
= 1.5*325[mm]
= **487.5**[mm] Sec. D.5.2.1

h_a<1.5*c_{a1} → 1000[mm]<1.5*325[mm] → **False**

h_a = 1.5*c_{a1}
= 1.5*325[mm]
= **487.5**[mm] Sec. D.5.2.1

IsCloseToThreeEdges → **False**

c_{a1} = c_{a1}
= **325**[mm] Sec. D.6.2.4

L_{Vc} = c_{a2Left} + $c_{a2Right}$
= 375[mm] + 487.5[mm]
= **862.5**[mm] Sec. RD.6.2.1

A_{Vc} = L_{Vc}*min(h_a, 1.5*c_{a1})
= 862.5[mm]*min(1000[mm], 1.5*325[mm])
= **4.20E+05**[mm2] Sec. RD.6.2.1

A_{Vco} = 4.5*c_{a1}^2
= 4.5*325[mm]^2
= **4.75E+05**[mm2] Eq. D-23

c_{a2}<1.5*c_{a1} → 375[mm]<1.5*325[mm] → **True**

$\psi_{ed,V}$ = 0.7 + 0.3*(c_{a2}/(1.5*c_{a1}))
= 0.7 + 0.3*(375[mm]/(1.5*325[mm]))
= **0.931** Eq. D-28

CrackedConcrete → **False**

$\psi_{c,V}$ = 1.4 Sec. D.6.2.7

h_a<1.5*c_{a1} → 1000[mm]<1.5*325[mm] → **False**

$\psi_{h,V}$ = 1 Sec. D.6.2.8

l_e = min(h_{ef}, 8*d_a)
= min(800[mm], 8*24[mm])

= **192**[mm] Sec. D.6.2.2

$V_b = (7*(l_e/d_a)^{0.2}*(d_a/(1\ [in]))^{1/2})*\lambda*(f_c/(1\ [psi]))^{1/2}*(c_{a1}/(1\ [in]))^{1.5}\ [lb]$
= $(7*(192[mm]/24[mm])^{0.2}*(24[mm]/(1\ [in]))^{1/2})*1*(50[N/mm2]/(1\ [psi]))^{1/2}*(325[mm]/(1\ [in]))^{1.5}\ [lb]$
= **178.81**[kN] Eq. D-24

$V_{cb} = (A_{Vc}/A_{Vco})*\psi_{ed,V}*\psi_{c,V}*\psi_{h,V}*V_b$
= (4.20E+05[mm2]/4.75E+05[mm2])*0.931*1.4*1*178.81[kN]
= **206.12**[kN] Eq. D-21

HighSeismicDesignCategory → **False**

$\phi V_{cb} = \phi*V_{cb}$
= 0.7*206.12[kN]
= **144.28**[kN] Sec. D.4.1.1

Breakout of group of anchors in shear	[KN]	188.23	44.01	CASE1	**0.23**	Sec. D.4.1.1

$A_{Vco} = 4.5*c_{a1}^2$
= $4.5*666.67[mm]^2$
= **2.00E+06**[mm2] Eq. D-23

$A_{Vc} = L_{Vc}*\min(h_a, 1.5*c_{a1})$
= 900[mm]*min(1000[mm], 1.5*666.67[mm])
= **9.00E+05**[mm2] Sec. RD.6.2.1

$A_{Vc} = \min(A_{Vc}, n*A_{Vco})$
= min(9.00E+05[mm2], 4*2.00E+06[mm2])
= **9.00E+05**[mm2] Sec. RD.6.2.1

$\psi_{ec,V} = \min(1/(1 + 2*e'_V/(3*c_{a1})), 1)$
= min(1/(1 + 2*0[mm]/(3*666.67[mm])), 1)
= **1** Eq. D-26

$c_{a2}<1.5*c_{a1}$ → 375[mm]<1.5*666.67[mm] → **True**

$\psi_{ed,V} = 0.7 + 0.3*(c_{a2}/(1.5*c_{a1}))$
= 0.7 + 0.3*(375[mm]/(1.5*666.67[mm]))
= **0.813** Eq. D-28

CrackedConcrete → **False**

$\psi_{c,V} = 1.4$ Sec. D.6.2.7

$h_a<1.5*c_{a1}$ → 1000[mm]<1.5*666.67[mm] → **False**

$\psi_{h,V} = 1$ Sec. D.6.2.8

$l_e = \min(h_{ef}, 8*d_a)$
= min(800[mm], 8*24[mm])
= **192**[mm] Sec. D.6.2.2

$V_b = (7*(l_e/d_a)^{0.2}*(d_a/(1\ [in]))^{1/2})*\lambda*(f_c/(1\ [psi]))^{1/2}*(c_{a1}/(1\ [in]))^{1.5}\ [lb]$
$= (7*(192[mm]/24[mm])^{0.2}*(24[mm]/(1\ [in]))^{1/2})*1*(50[N/mm2]/(1\ [psi]))^{1/2}*(666.67[mm]/(1\ [in]))^{1.5}\ [lb]$
= **525.33**[kN] Eq. D-24

$V_{cbg} = (A_{Vc}/A_{Vco})*\psi_{ec,V}*\psi_{ed,V}*\psi_{c,V}*\psi_{h,V}*V_b$
= (9.00E+05[mm2]/2.00E+06[mm2])*1*0.813*1.4*1*525.33[kN]
= **268.9**[kN] Eq. D-22

HighSeismicDesignCategory → **False**

$\phi V_{cbg} = \phi * V_{cbg}$
= 0.7*268.9[kN]
= **188.23**[kN] Sec. D.4.1.1

Pryout of anchor in shear	[KN]	571.59	11.00	CASE1	**0.02**	Eq. D-4, Sec. D.4.1.1

h_{ef}<2.5 [in] → 800[mm]<2.5 [in] → **False**

$k_{cp} = 2$ Sec. D.6.3.1

$c_{a1Left}<1.5*h_{ef}$ → 375[mm]<1.5*800[mm] → **True**

$c_{a1Left} = c_{a1Left}$
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}<1.5*h_{ef}$ → 525[mm]<1.5*800[mm] → **True**

$c_{a1Right} = c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

$c_{a2Top}<1.5*h_{ef}$ → 925[mm]<1.5*800[mm] → **True**

$c_{a2Top} = c_{a2Top}$
= **925**[mm] Sec. D.5.2.1

$c_{a2Bot}<1.5*h_{ef}$ → 325[mm]<1.5*800[mm] → **True**

$c_{a2Bot} = c_{a2Bot}$
= **325**[mm] Sec. D.5.2.1

IsCloseToThreeEdges → **True**

$h_{ef} = c_{amax}/1.5$
= 925[mm]/1.5
= **616.67**[mm] Sec. D.5.2.3

$c_{a1Left}<1.5*h_{ef}$ → 375[mm]<1.5*616.67[mm] → **True**

$c_{a1Left} = c_{a1Left}$
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}$<1.5*h_{ef} → 525[mm]<1.5*616.67[mm] → **True**

$c_{a1Right}$ = $c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*616.67[mm] → **False**

c_{a2Top} = 1.5*h_{ef}
= 1.5*616.67[mm]
= **925**[mm] Sec. D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

c_{a2Bot} = c_{a2Bot}
= **325**[mm] Sec. D.5.2.1

A_{Nc} = (c_{a1Left} + $c_{a1Right}$)*(c_{a2Top} + c_{a2Bot})
= (375[mm] + 525[mm])*(925[mm] + 325[mm])
= **1.13E+06**[mm2] Sec. RD.5.2.1

A_{Nco} = 9*h_{ef}^2
= 9*616.67[mm]^2
= **3.42E+06**[mm2] Eq. D-6

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*616.67[mm])
= **0.805** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (616.67[mm]> = 11 [in]) and (616.67[mm]<
= 25 [in]) → **True**

N_b = 16*λ*(f_c/(1 [psi]))$^{1/2}$*(h_{ef}/(1 [in]))$^{(5/3)}$ [lb]
= 16*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(616.67[mm]/(1 [in]))$^{(5/3)}$ [lb]
= **1233.75**[kN] Eq. D-8

N_{cb} = (A_{Nc}/A_{Nco})*$\psi_{ed,N}$*$\psi_{c,N}$*$\psi_{cp,N}$*N_b
= (1.13E+06[mm2]/3.42E+06[mm2])*0.805*1.25*1*1233.75[kN]

= **408.28**[kN] Eq. D-4

$V_{cp} = k_{cp} * N_{cb}$
= 2*408.28[kN]
= **816.56**[kN] Eq. D-30

HighSeismicDesignCategory → **False**

$\phi V_{cp} = \phi * V_{cp}$
= 0.7*816.56[kN]
= **571.59**[kN] Sec. D.4.1.1

Pryout of group of anchors in shear	[KN]	943.24	44.01	CASE1	**0.05**	Eq. D-5, Sec. D.4.1.1

h_{ef}<2.5 [in] → 800[mm]<2.5 [in] → **False**

$k_{cp} = 2$ Sec. D.6.3.1

$A_{Nco} = 9 * h_{ef}^2$
= 9*250[mm]²
= **5.63E+05**[mm2] Eq. D-6

$A_{Nc} = \min(A_{Nc}, n * A_{Nco})$
= min(1.13E+06[mm2], 4*5.63E+05[mm2])
= **1.13E+06**[mm2] Sec. D.5.2.1

$\psi_{ec,Ny} = \min(1/(1 + 2 * e'_N/(3 * h_{ef})), 1)$
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,Nx} = \min(1/(1 + 2 * e'_N/(3 * h_{ef})), 1)$
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,N} = \psi_{ec,Nx} * \psi_{ec,Ny}$
= 1*1
= **1** Eq. D-9

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*250[mm] → **True**

$\psi_{ed,N} = 0.7 + 0.3 * c_{a,min}/(1.5 * h_{ef})$
= 0.7 + 0.3*325[mm]/(1.5*250[mm])
= **0.96** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N} = 1.25$ Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N} = 1$ Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) $\rightarrow$ (True) and (True) and (250[mm]> = 11 [in]) and (250[mm]<
= 25 [in]) $\rightarrow$ **False**

$N_b = k_c*\lambda*(f_c/(1\ [psi]))^{1/2}*(h_{ef}/(1\ [in]))^{1.5}$ [lb]
= 24*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(250[mm]/(1 [in]))$^{1.5}$ [lb]
= **280.73**[kN] Eq. D-7

$N_{cbg} = (A_{Nc}/A_{Nco})*\psi_{ec,N}*\psi_{ed,N}*\psi_{c,N}*\psi_{cp,N}*N_b$
= (1.13E+06[mm2]/5.63E+05[mm2])*1*0.96*1.25*1*280.73[kN]
= **673.74**[kN] Eq. D-5

$V_{cpg} = k_{cp}*N_{cbg}$
= 2*673.74[kN]
= **1347.48**[kN] Eq. D-31

HighSeismicDesignCategory $\rightarrow$ **False**

$\phi V_{cpg} = \phi*V_{cpg}$
= 0.7*1347.48[kN]
= **943.24**[kN] Sec. D.4.1.1

Interaction of tensile and shear forces	[KN]	1.20	0.99	CASE1	**0.82**	Eq. D-3, Eq. D-4, Sec. D.4.1.1, Eq. D-5, Eq. D-20, Sec. D.6.1.3, Eq. D-32

$A_{se} = \pi/4.0*(d_a - 0.9743\ [in]/n_t)^2$
= π/4.0*(24[mm] - 0.9743 [in]/8)^2
= **343.29**[mm2] Sec. D.5.1.1, D.6.1.2

$f_{uta} = \min(f_{uta}, 1.9*f_{ya}, 125\ [ksi])$
= min(399.89[N/mm2], 1.9*248.21[N/mm2], 125 [ksi])
= **399.89**[N/mm2] Sec. D.5.1.2

$\phi N_{sa} = \phi*n*A_{se,N}*f_{uta}$
= 0.75*1*343.29[mm2]*399.89[N/mm2]
= **102.96**[kN] Eq. D-3

$c_{a1Left}<1.5*h_{ef}$ $\rightarrow$ 375[mm]<1.5*800[mm] $\rightarrow$ **True**

$c_{a1Left} = c_{a1Left}$
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}<1.5*h_{ef}$ $\rightarrow$ 525[mm]<1.5*800[mm] $\rightarrow$ **True**

$c_{a1Right} = c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*800[mm] → **True**

c_{a2Top} = c_{a2Top}
= **925**[mm] Sec. D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*800[mm] → **True**

c_{a2Bot} = c_{a2Bot}
= **325**[mm] Sec. D.5.2.1

IsCloseToThreeEdges → **True**

h_{ef} = c_{amax}/1.5
= 925[mm]/1.5
= **616.67**[mm] Sec. D.5.2.3

c_{a1Left}<1.5*h_{ef} → 375[mm]<1.5*616.67[mm] → **True**

c_{a1Left} = c_{a1Left}
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}$<1.5*h_{ef} → 525[mm]<1.5*616.67[mm] → **True**

$c_{a1Right}$ = $c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*616.67[mm] → **False**

c_{a2Top} = 1.5*h_{ef}
= 1.5*616.67[mm]
= **925**[mm] Sec. D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

c_{a2Bot} = c_{a2Bot}
= **325**[mm] Sec. D.5.2.1

A_{Nc} = (c_{a1Left} + $c_{a1Right}$)*(c_{a2Top} + c_{a2Bot})
= (375[mm] + 525[mm])*(925[mm] + 325[mm])
= **1.13E+06**[mm2] Sec. RD.5.2.1

A_{Nco} = 9*h_{ef}^2
= 9*616.67[mm]^2
= **3.42E+06**[mm2] Eq. D-6

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*616.67[mm])
= **0.805** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (616.67[mm]> = 11 [in]) and (616.67[mm]<
= 25 [in]) → **True**

N_b = 16*λ*(f_c/(1 [psi]))$^{1/2}$*(h_{ef}/(1 [in]))$^{(5/3)}$ [lb]
= 16*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(616.67[mm]/(1 [in]))$^{(5/3)}$ [lb]
= **1233.75**[kN] Eq. D-8

N_{cb} = (A_{Nc}/A_{Nco})*$\psi_{ed,N}$*$\psi_{c,N}$*$\psi_{cp,N}$*N_b
= (1.13E+06[mm2]/3.42E+06[mm2])*0.805*1.25*1*1233.75[kN]
= **408.28**[kN] Eq. D-4

HighSeismicDesignCategory → **False**

ϕN_{cb} = ϕ*N_{cb}
= 0.7*408.28[kN]
= **285.8**[kN] Sec. D.4.1.1

A_{Nco} = 9*h_{ef}^2
= 9*250[mm]^2
= **5.63E+05**[mm2] Eq. D-6

A_{Nc} = min(A_{Nc}, n*A_{Nco})
= min(1.13E+06[mm2], 4*5.63E+05[mm2])
= **1.13E+06**[mm2] Sec. D.5.2.1

$\psi_{ec,Ny}$ = min(1/(1 + 2*e'$_N$/(3*h_{ef})), 1)
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,Nx}$ = min(1/(1 + 2*e'$_N$/(3*h_{ef})), 1)
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,N}$ = $\psi_{ec,Nx}$*$\psi_{ec,Ny}$
= 1*1
= **1** Eq. D-9

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*250[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*250[mm])

= **0.96** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (250[mm]> = 11 [in]) and (250[mm]<
= 25 [in]) → **False**

$N_b = k_c*\lambda*(f_c/(1\ [psi]))^{1/2}*(h_{ef}/(1\ [in]))^{1.5}\ [lb]$
$= 24*1*(50[N/mm2]/(1\ [psi]))^{1/2}*(250[mm]/(1\ [in]))^{1.5}\ [lb]$
= **280.73**[kN] Eq. D-7

$N_{cbg} = (A_{Nc}/A_{Nco})*\psi_{ec,N}*\psi_{ed,N}*\psi_{c,N}*\psi_{cp,N}*N_b$
= (1.13E+06[mm2]/5.63E+05[mm2])*1*0.96*1.25*1*280.73[kN]
= **673.74**[kN] Eq. D-5

HighSeismicDesignCategory → **False**

$\phi N_{cbg} = \phi*N_{cbg}$
= 0.7*673.74[kN]
= **471.62**[kN] Sec. D.4.1.1

$A_{brg} = F^2 - A_g$
$= 36[mm]^2 - 452[mm2]$
= **844**[mm2]

IsHeadedBolt → **True**

$N_p = 8*A_{brg}*f_c$
= 8*844[mm2]*50[N/mm2]
= **337.6**[kN] Eq. D-15

CrackedConcrete → **False**

$\psi_{c,P}$ = 1.4 Sec. D.5.3.6

$N_{pn} = \psi_{c,P}*N_p$
= 1.4*337.6[kN]
= **472.64**[kN] Eq. D-14

HighSeismicDesignCategory → **False**

$\phi N_{pn} = \phi*N_{pn}$
= 0.7*472.64[kN]

= **330.85**[kN] Sec. D.4.1.1

SideFaceBlowoutApply = h_{ef}>2.5*c_{a1}
= 800[mm]>2.5*325[mm]
= **False** Sec. D.5.4.1

A_{se} = π/4.0*(d_a - 0.9743 [in]/n_t)^2
= π/4.0*(24[mm] - 0.9743 [in]/8)^2
= **343.29**[mm2] Sec. D.5.1.1, D.6.1.2

f_{uta} = min(f_{uta}, 1.9*f_{ya}, 125 [ksi])
= min(399.89[N/mm2], 1.9*248.21[N/mm2], 125 [ksi])
= **399.89**[N/mm2] Sec. D.5.1.2

HasGroutPad → **True**

ϕV_{sa} = 0.8*ϕ*0.6*n*$A_{se,V}$*f_{uta}
= 0.8*0.65*0.6*1*343.29[mm2]*399.89[N/mm2]
= **42.83**[kN] Eq. D-20, Sec. D.6.1.3

c_{a2Left}<1.5*c_{a1} → 375[mm]<1.5*325[mm] → **True**

c_{a2Left} = c_{a2Left}
= **375**[mm] Sec. D.6.2.1

$c_{a2Right}$<1.5*c_{a1} → 525[mm]<1.5*325[mm] → **False**

$c_{a2Right}$ = 1.5*c_{a1}
= 1.5*325[mm]
= **487.5**[mm] Sec. D.5.2.1

h_a<1.5*c_{a1} → 1000[mm]<1.5*325[mm] → **False**

h_a = 1.5*c_{a1}
= 1.5*325[mm]
= **487.5**[mm] Sec. D.5.2.1

IsCloseToThreeEdges → **False**

c_{a1} = c_{a1}
= **325**[mm] Sec. D.6.2.4

L_{Vc} = c_{a2Left} + $c_{a2Right}$
= 375[mm] + 487.5[mm]
= **862.5**[mm] Sec. RD.6.2.1

A_{Vc} = L_{Vc}*min(h_a, 1.5*c_{a1})
= 862.5[mm]*min(1000[mm], 1.5*325[mm])

$= \mathbf{4.20E+05}$[mm2] Sec. RD.6.2.1

$A_{Vco} = 4.5 * c_{a1}^2$
$= 4.5 * 325$[mm]^2
$= \mathbf{4.75E+05}$[mm2] Eq. D-23

$c_{a2} < 1.5 * c_{a1} \rightarrow 375$[mm]$< 1.5 * 325$[mm] $\rightarrow$ **True**

$\psi_{ed,V} = 0.7 + 0.3 * (c_{a2}/(1.5 * c_{a1}))$
$= 0.7 + 0.3 * (375$[mm]$/(1.5 * 325$[mm]$))$
$= \mathbf{0.931}$ Eq. D-28

CrackedConcrete $\rightarrow$ **False**

$\psi_{c,V} = 1.4$ Sec. D.6.2.7

$h_a < 1.5 * c_{a1} \rightarrow 1000$[mm]$< 1.5 * 325$[mm] $\rightarrow$ **False**

$\psi_{h,V} = 1$ Sec. D.6.2.8

$l_e = \min(h_{ef}, 8 * d_a)$
$= \min(800$[mm]$, 8 * 24$[mm]$)$
$= \mathbf{192}$[mm] Sec. D.6.2.2

$V_b = (7 * (l_e/d_a)^{0.2} * (d_a/(1$ [in]$))^{1/2}) * \lambda * (f_c/(1$ [psi]$))^{1/2} * (c_{a1}/(1$ [in]$))^{1.5}$ [lb]
$= (7 * (192$[mm]$/24$[mm]$)^{0.2} * (24$[mm]$/(1$ [in]$))^{1/2}) * 1 * (50$[N/mm2]$/(1$ [psi]$))^{1/2} * (325$[mm]$/$
$(1$ [in]$))^{1.5}$ [lb]
$= \mathbf{178.81}$[kN] Eq. D-24

$V_{cb} = (A_{Vc}/A_{Vco}) * \psi_{ed,V} * \psi_{c,V} * \psi_{h,V} * V_b$
$= (4.20E+05$[mm2]$/4.75E+05$[mm2]$) * 0.931 * 1.4 * 1 * 178.81$[kN]
$= \mathbf{206.12}$[kN] Eq. D-21

HighSeismicDesignCategory $\rightarrow$ **False**

$\phi V_{cb} = \phi * V_{cb}$
$= 0.7 * 206.12$[kN]
$= \mathbf{144.28}$[kN] Sec. D.4.1.1

$A_{Vco} = 4.5 * c_{a1}^2$
$= 4.5 * 666.67$[mm]^2
$= \mathbf{2.00E+06}$[mm2] Eq. D-23

$A_{Vc} = L_{Vc} * \min(h_a, 1.5 * c_{a1})$
$= 900$[mm]$ * \min(1000$[mm]$, 1.5 * 666.67$[mm]$)$
$= \mathbf{9.00E+05}$[mm2] Sec. RD.6.2.1

$A_{Vc} = \min(A_{Vc}, n * A_{Vco})$
$= \min(9.00E+05$[mm2]$, 4 * 2.00E+06$[mm2]$)$
$= \mathbf{9.00E+05}$[mm2] Sec. RD.6.2.1

$\psi_{ec,V} = \min(1/(1 + 2 * e'_V/(3 * c_{a1})), 1)$
$= \min(1/(1 + 2 * 0$[mm]$/(3 * 666.67$[mm]$)), 1)$

= **1** Eq. D-26

c_{a2}<1.5*c_{a1} → 375[mm]<1.5*666.67[mm] → **True**

$\psi_{ed,V}$ = 0.7 + 0.3*(c_{a2}/(1.5*c_{a1}))
= 0.7 + 0.3*(375[mm]/(1.5*666.67[mm]))
= **0.813** Eq. D-28

CrackedConcrete → **False**

$\psi_{c,V}$ = 1.4 Sec. D.6.2.7

h_a<1.5*c_{a1} → 1000[mm]<1.5*666.67[mm] → **False**

$\psi_{h,V}$ = 1 Sec. D.6.2.8

l_e = min(h_{ef}, 8*d_a)
= min(800[mm], 8*24[mm])
= **192**[mm] Sec. D.6.2.2

V_b = (7*(l_e/d_a)$^{0.2}$*(d_a/(1 [in]))$^{1/2}$)*λ*(f_c/(1 [psi]))$^{1/2}$*(c_{a1}/(1 [in]))$^{1.5}$ [lb]
= (7*(192[mm]/24[mm])$^{0.2}$*(24[mm]/(1 [in]))$^{1/2}$)*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(666.67[mm]/(1 [in]))$^{1.5}$ [lb]
= **525.33**[kN] Eq. D-24

V_{cbg} = (A_{Vc}/A_{Vco})*$\psi_{ec,V}$*$\psi_{ed,V}$*$\psi_{c,V}$*$\psi_{h,V}$*V_b
= (9.00E+05[mm2]/2.00E+06[mm2])*1*0.813*1.4*1*525.33[kN]
= **268.9**[kN] Eq. D-22

HighSeismicDesignCategory → **False**

ϕV_{cbg} = ϕ*V_{cbg}
= 0.7*268.9[kN]
= **188.23**[kN] Sec. D.4.1.1

h_{ef}<2.5 [in] → 800[mm]<2.5 [in] → **False**

k_{cp} = 2 Sec. D.6.3.1

c_{a1Left}<1.5*h_{ef} → 375[mm]<1.5*800[mm] → **True**

c_{a1Left} = c_{a1Left}
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}$<1.5*h_{ef} → 525[mm]<1.5*800[mm] → **True**

$c_{a1Right}$ = $c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*800[mm] → **True**

c_{a2Top} = c_{a2Top}

= **925**[mm] Sec. D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*800[mm] → **True**

c_{a2Bot} = c_{a2Bot}
= **325**[mm] Sec. D.5.2.1

IsCloseToThreeEdges → **True**

h_{ef} = c_{amax}/1.5
= 925[mm]/1.5
= **616.67**[mm] Sec. D.5.2.3

c_{a1Left}<1.5*h_{ef} → 375[mm]<1.5*616.67[mm] → **True**

c_{a1Left} = c_{a1Left}
= **375**[mm] Sec. D.5.2.1

$c_{a1Right}$<1.5*h_{ef} → 525[mm]<1.5*616.67[mm] → **True**

$c_{a1Right}$ = $c_{a1Right}$
= **525**[mm] Sec. D.5.2.1

c_{a2Top}<1.5*h_{ef} → 925[mm]<1.5*616.67[mm] → **False**

c_{a2Top} = 1.5*h_{ef}
= 1.5*616.67[mm]
= **925**[mm] Sec. D.5.2.1

c_{a2Bot}<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

c_{a2Bot} = c_{a2Bot}
= **325**[mm] Sec. D.5.2.1

A_{Nc} = (c_{a1Left} + $c_{a1Right}$)*(c_{a2Top} + c_{a2Bot})
= (375[mm] + 525[mm])*(925[mm] + 325[mm])
= **1.13E+06**[mm2] Sec. RD.5.2.1

A_{Nco} = 9*h_{ef}^2
= 9*616.67[mm]^2
= **3.42E+06**[mm2] Eq. D-6

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*616.67[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*616.67[mm])
= **0.805** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (616.67[mm]> = 11 [in]) and (616.67[mm]<
= 25 [in]) → **True**

N_b = 16*λ*(f_c/(1 [psi]))$^{1/2}$*(h_{ef}/(1 [in]))$^{(5/3)}$ [lb]
= 16*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(616.67[mm]/(1 [in]))$^{(5/3)}$ [lb]
= **1233.75**[kN] Eq. D-8

N_{cb} = (A_{Nc}/A_{Nco})*$\psi_{ed,N}$*$\psi_{c,N}$*$\psi_{cp,N}$*N_b
= (1.13E+06[mm2]/3.42E+06[mm2])*0.805*1.25*1*1233.75[kN]
= **408.28**[kN] Eq. D-4

V_{cp} = k_{cp}*N_{cb}
= 2*408.28[kN]
= **816.56**[kN] Eq. D-30

HighSeismicDesignCategory → **False**

ϕV_{cp} = ϕ*V_{cp}
= 0.7*816.56[kN]
= **571.59**[kN] Sec. D.4.1.1

h_{ef}<2.5 [in] → 800[mm]<2.5 [in] → **False**

k_{cp} = 2 Sec. D.6.3.1

A_{Nco} = 9*h_{ef}^2
= 9*250[mm]^2
= **5.63E+05**[mm2] Eq. D-6

A_{Nc} = min(A_{Nc}, n*A_{Nco})
= min(1.13E+06[mm2], 4*5.63E+05[mm2])
= **1.13E+06**[mm2] Sec. D.5.2.1

$\psi_{ec,Ny}$ = min(1/(1 + 2*e'$_N$/(3*h_{ef})), 1)
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,Nx}$ = min(1/(1 + 2*e'$_N$/(3*h_{ef})), 1)
= min(1/(1 + 2*0[mm]/(3*250[mm])), 1)
= **1** Eq. D-9

$\psi_{ec,N}$ = $\psi_{ec,Nx}$*$\psi_{ec,Ny}$
= 1*1
= **1** Eq. D-9

$c_{a,min}$<1.5*h_{ef} → 325[mm]<1.5*250[mm] → **True**

$\psi_{ed,N}$ = 0.7 + 0.3*$c_{a,min}$/(1.5*h_{ef})
= 0.7 + 0.3*325[mm]/(1.5*250[mm])
= **0.96** Eq. D-11

CrackedConcrete → **False**

$\psi_{c,N}$ = 1.25 Sec. D.5.2.6

IsCastInPlaceAnchor → **True**

$\psi_{cp,N}$ = 1 Sec. D.5.2.7

IsCastInPlaceAnchor → **True**

k_c = 24 Sec. D.5.2.2

(IsCastInPlaceAnchor) and (IsHeadedBolt) and (h_{ef}>=11 [in]) and (h_{ef}<=25 [in]) → (True) and (True) and (250[mm]> = 11 [in]) and (250[mm]<
= 25 [in]) → **False**

N_b = k_c*λ*(f_c/(1 [psi]))$^{1/2}$*(h_{ef}/(1 [in]))$^{1.5}$ [lb]
= 24*1*(50[N/mm2]/(1 [psi]))$^{1/2}$*(250[mm]/(1 [in]))$^{1.5}$ [lb]
= **280.73**[kN] Eq. D-7

N_{cbg} = (A_{Nc}/A_{Nco})*$\psi_{ec,N}$*$\psi_{ed,N}$*$\psi_{c,N}$*$\psi_{cp,N}$*N_b
= (1.13E+06[mm2]/5.63E+05[mm2])*1*0.96*1.25*1*280.73[kN]
= **673.74**[kN] Eq. D-5

V_{cpg} = k_{cp}*N_{cbg}
= 2*673.74[kN]
= **1347.48**[kN] Eq. D-31

HighSeismicDesignCategory → **False**

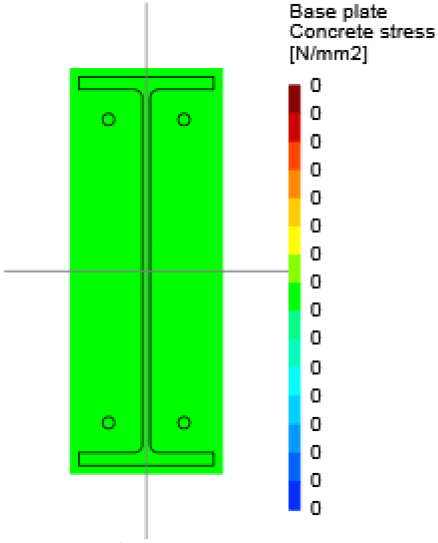
ϕV_{cpg} = ϕ*V_{cpg}
= 0.7*1347.48[kN]
= **943.24**[kN] Sec. D.4.1.1

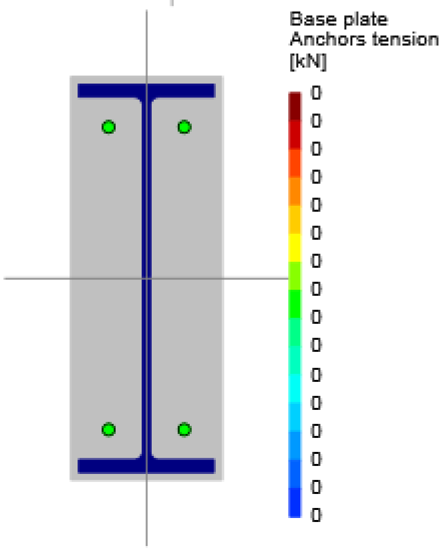
(N_{ua}>0.2*ϕN_n) and (V_{ua}>0.2*ϕV_n) → (75.1[kN]>0.2*102.96[kN]) and (11[kN]>0.2*42.83[kN]) → **True**

TensionShearInteraction = N_{ua}/ϕN_n + V_{ua}/ϕV_n
= 75.1[kN]/102.96[kN] + 11[kN]/42.83[kN]
= **0.986** Eq. D-32

Ratio	**0.82**
Global critical strength ratio	**0.99**

Major axis
Maximum compression (CASE2)



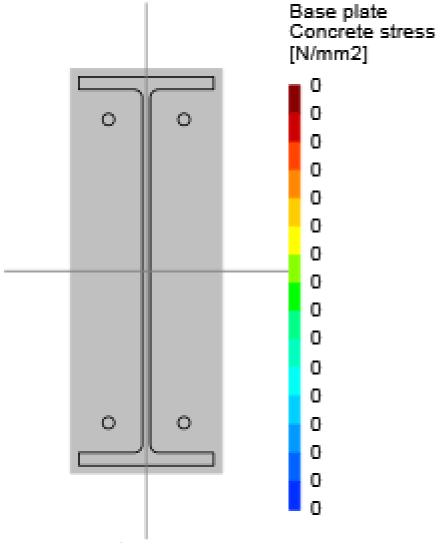


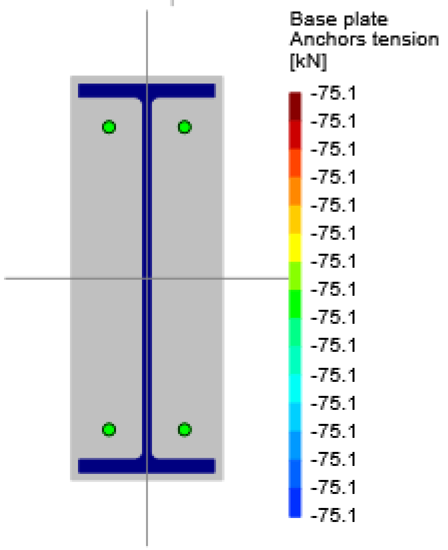
Maximum bearing pressure	0.00	[N/mm2]
Minimum bearing pressure	0.00	[N/mm2]
Maximum anchor tension	0.00	[KN]
Minimum anchor tension	0.00	[KN]
Neutral axis angle	0.00	
Bearing length	1E33	[mm]

Anchors tensions

Anchor	Transverse [mm]	Longitudinal [mm]	Shear [KN]	Tension [KN]
1	-75.00	-300.00	-11.00	0.00
2	-75.00	300.00	-11.00	0.00
3	75.00	300.00	-11.00	0.00
4	75.00	-300.00	-11.00	0.00

Maximum tension (CASE1)





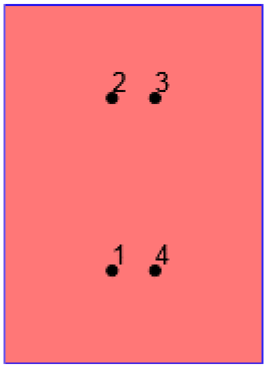
Maximum bearing pressure	0.00	[N/mm2]
Minimum bearing pressure	0.00	[N/mm2]
Maximum anchor tension	75.10	[KN]
Minimum anchor tension	75.10	[KN]
Neutral axis angle	0.00	
Bearing length	-1E33	[mm]

Anchors tensions

Anchor	Transverse [mm]	Longitudinal [mm]	Shear [KN]	Tension [KN]
1	-75.00	-300.00	11.00	75.10
2	-75.00	300.00	11.00	75.10
3	75.00	300.00	11.00	75.10
4	75.00	-300.00	11.00	75.10

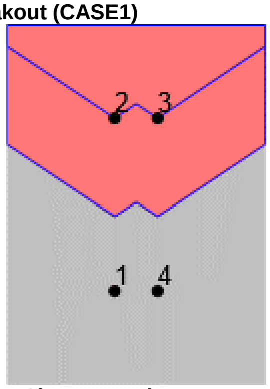
Major axis

Results for tensile breakout (CASE1)



Group	Area [mm2]	Tension [KN]	Anchors
1	1125000.00	300.40	1, 2, 3, 4

Results for shear breakout (CASE1)



Group	Area [mm2]	Shear [KN]	Anchors
1	900000.00	44.01	1, 2, 3, 4
2	438750.00	22.01	2, 3

NOTATION

A_1: Base plate area
A_2: Maximum area of portion of the concrete supporting surface that is geometrically similar to and concentric with the load area
A_w: Effective area of the weld
A_2/A_1: Ratio between the concrete support area and the base plate area
B: Base plate design width
b_c: Width of column section
b_{eff}: Effective width of the compression block
B_{eff}: Controlling ffective width
c: Distance to weld group
d_c: Column depth
D: Number of sixteenths of an inch in the weld size
f_a: Axial stress on welds
f_b: Bending stress on welds
f'_c: Specified compressive strength of concrete
f: Combined stress on welds
F_{EXX}: Electrode classification number
f_p: Uniformly bearing stress under base plate
$f_{p, max}$: Maximum uniformly bearing stress under base plate
f_v: Vertical shear force on weld
F_w: Nominal strength of the weld metal per unit area
F_y: Specified minimum yield stress
I: Inertia of weld group
L: Distance from the anchor rod to the column
l: Critical base plate cantilever dimension
L: Length of weld

L_{shear}: Length of weld receiving shear
λ: Auxiliary variable to calculate the critical base plate cantilever dimension
LoadAngleFactor: Load angle factor
M: Bending required
m: Base plate bearing interface cantilever direction parallel to moment direction
M_{pl}: Plate bending moment per unit width
M_{pT}: Plate bending moment per unit width at tension unstiffened strip interface
M_{strip}: Maximum bending moment at the strip
Maximum weld load: Maximum weld load
N: Base plate design length
n: Base plate bearing interface cantilever direction perpendicular to moment direction
n': Yield line theory cantilever distance from column web or column flange
N_{cs}: Length of the concrete supporting surface or pier parallel to moment design direction
P: Required axial force
P_p: Nominal bearing stress
ϕ: Design factors
ϕM_n: Design or allowable strength per unit length
ϕR_w: Fillet weld capacity per unit length
T: Anchor rod tensile strength required
t_p: Plate thickness
θ: Load angle
V: Shear load
w_{min}: Minimum weld size required
X: Auxiliary variable to calculate the critical base plate cantilever dimension
A_{brg}: Net bearing area of the head of stud or anchor bolt
A_g: Gross area of anchor
A_{Nc}: Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension
A_{Nco}: Projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing
A_{se}: Effective cross-sectional area of anchor
$A_{se,N}$: Effective cross-sectional area of anchor in tension
$A_{se,V}$: Effective cross-sectional area of anchor in shear
A_{Vc}: Projected concrete failure area of a single anchor or group of anchors , for calculation of strength in shear
A_{Vco}: Projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness
c_{a1}: Distance from the anchor center to the concrete edge
c_{a1Left}: Distance from the anchor center to the left edge of the concrete base
$c_{a1Right}$: Distance from the anchor center to the right edge of the concrete base
c_{a2}: Distance from the anchor center to the concrete edge in perpendicular direction
c_{a2Bot}: Distance from the anchor center to the bottom edge of the concrete base
c_{a2Left}: Distance from the anchor center to the left edge of the concrete base
$c_{a2Right}$: Distance from the anchor center to the right edge of the concrete base
c_{a2Top}: Distance from the anchor center to the top edge of the concrete base
c_{amax}: Maximum distance from center of an anchor shaft to the edge of concrete
$c_{a,min}$: Minimum distance from center of an anchor shaft to the edge of concrete
Cover: Concrete cover
CrackedConcrete: Cracked concrete at service loads
d_a: Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt
e'_N: Distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension
e'_V: Distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction
F: Distance between head flat sides
f_c: Specified compressive strength of concrete
f_{uta}: Specified tensile strength of anchor steel
f_{ya}: Specified yield strength of anchor steel
h_a: Thickness of member in which an anchor is located, measured parallel to anchor axis
h_{ef}: Effective embedment depth of anchor
HasGroutPad: Has grout pad
HighSeismicDesignCategory: High seismic design category (i.e. C, D, E or F)
IsCastInPlaceAnchor: Is cast in place anchor
IsCloseToThreeEdges: Anchor is close to three or more edges
IsConcreteCastAgainstEarth: Is concrete cast against and permanently exposed to earth
IsHeadedBolt: Is anchor headed stud
k_c: Coefficient for concrete pry out basic strength
k_{cp}: Coefficient for pry out strength
l_e: Load-bearing length of the anchor for shear
L_{Vc}: Projected concrete failure length of a single anchor or group of anchors , for calculation of strength in shear

λ: Lightweight concrete modification factor
n: Number of anchors in the group
N_b: Basic concrete breakout strength in tension of a single anchor in cracked concrete
N_{cb}: Nominal concrete breakout strength in tension of a single anchor
N_{cbg}: Nominal concrete breakout strength in tension of a group of anchors
N_p: Pullout strength in tension of a single anchor in cracked concrete
N_{pn}: Nominal pullout strength of a single anchor in tension
n_t: Number of threads per inch
N_{ua}: Factored tensile force applied to anchor or group of anchors
ϕ: Strength reduction factor
ϕN_{cb}: Concrete breakout strength in tension of a single anchor
ϕN_{cbg}: Concrete breakout strength in tension of a group of anchors
ϕN_n: Tension strength
ϕN_{pn}: Pullout strength in tension of a single anchor
ϕN_{sa}: Strength of a single anchor or group of anchors in tension
ϕV_{cb}: Concrete breakout strength in shear of a single anchor
ϕV_{cbg}: Concrete breakout strength in shear of a group of anchors
ϕV_{cp}: Concrete pryout strength of a single anchor
ϕV_{cpg}: Concrete pryout strength of a group of anchors
ϕV_n: Shear strength
ϕV_{sa}: Strength in shear of a single anchor or group of anchors as governed by the steel strength
$\psi_{c,N}$: Factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete
$\psi_{c,P}$: Factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete
$\psi_{cp,N}$: Factor used to modify tensile strength of postinstalled anchors intended for use in uncracked concrete without supplementary reinforcement
$\psi_{c,V}$: Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement
$\psi_{ec,N}$: Factor used to modify tensile strength of anchors based on eccentricity of applied loads
$\psi_{ec,Nx}$: Factor used to modify tensile strength of anchors based on eccentricity in x axis of applied loads
$\psi_{ec,Ny}$: Factor used to modify tensile strength of anchors based on eccentricity in y axis of applied loads
$\psi_{ec,V}$: Factor used to modify shear strength of anchors based on eccentricity of applied loads
$\psi_{ed,N}$: Factor used to modify tensile strength of anchors based on proximity to edges of concrete member
$\psi_{ed,V}$: Factor used to modify shear strength of anchors based on proximity to edges of concrete member
$\psi_{h,V}$: Factor used to modify shear strength of anchors located in concrete members with ha < 1.5ca1
s_{min}: Center-to-center anchor minimum spacing
SideFaceBlowoutApply: Side-face blowout apply
TensionShearInteraction: Result from tension-shear interaction
V_b: Basic concrete breakout strength in shear of a single anchor in cracked concrete
V_{cb}: Concrete nominal breakout strength in shear of a single anchor
V_{cbg}: Concrete nominal breakout strength in shear of a group of anchors
V_{cp}: Nominal pryout strength of a anchor in shear
V_{cpg}: Nominal pryout strength of a group of anchor in shear