INSDAG YEARBOOK
2021-2022

INSTITUTE FOR STEEL DEVELOPMENT AND GROWTH
(INSDAG)

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INSDAG	Steel is the backbone of all industries and the basic ingredient for growth and development of a country. Traditionally, the fortunes of the steel industry have been linked to the economic cycle of the country. Per capita consumption of steel speaks volumes about the relative position of the country on the development frontier. In India the per capita consumption of steel stands low compared to developed and developing countries. Moreover, steel is completely recyclable and environment friendly. Hence, a large potential exists in furthering the usage of steel in various segments of industry. Institute of Steel Development and Growth (INSDAG), a non-profit making, member based organization established by Ministry of Steel and the major steel producers of the country. The Institute primarily works towards the development of advanced design methodologies & technical marketing by expanding applications of steel in different segments of industry, upgrading skills & know-how, creating awareness amongst potential users and communicating the benefits of steel. Our founding members are SAIL, Tata Steel Ltd., RINL, JSW Steel Ltd., and Arcelor Mittal Nippon Steel India Limited (AM / NS) apart from Ministry of Steel. INSDAG has got very good networking among the member organisations/professionals for exchange of ideas. The Institute is registered as a "Society" under Societies Registration Act of West Bengal. Director General looks after the daily affairs of the Institute and Executive Council provides guidance and direction. Two other functional committees namely the Working Group and Project Review Committee provide administrative and technical guidance respectively. The Institute has defined its mission, role, and functions and has evolved its short, medium and long term Activity Plans. The Institute primarily works towards the development of technology in steel usage and the market for the steel fraternity. Some of its roles are: ➢ Creating awareness amongst potential users on affordability of steel. ➢ Bringing out technical publication on steel applications. ➢ Providing technical advisory services on materials, construction practices etc. ➢ Upgrading the skills of work force by refresher courses / training programmers. ➢ Communicating the benefits of steel through life cycle cost studies. ➢ Providing requisite thrust to increase steel consumption in rural areas. ➢ Assisting in the development of ancillary industries for creating new market.

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Mission Statement

To work in unison for all the stakeholders in the steel industry so as to evolve ways and means for more efficient usage of steel and provide optimum value to the customer

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PREFACE

INSDAG Yearbook 2021-2022 contains the technical articles from experts in steel industry.

The document contains article like Comparative Analysis and Optimization of Steel In Pre-Engineered Building by Prof. A.J. Shah, Associate Professor, Structural Engineering, and Kalpesh Jeengar, PG Scholar, Structural Engineering, DoCE, SVNIT, Surat; Efficiency of Steel Diagrid Buildings With Buckling-Restrained-Braces under Earthquake Load by Dr. Soumya Bhattacharjya, Associate Professor, Department of Civil Engineering, and Debtanu Karmakar, Master's student, Department of Civil Engineering, Indian Institute of Engineering Science and Technology, Shibpur; Buildings in Severe Earthquake Zones Made of Structural Steel Hollow And Plate Members By Arup Saha Chaudhuri Associate Professor, Civil Engineering Department, Techno Main Salt Lake, Kolkata and Avijit Ghosh M.Tech (Structure), Civil Engineering Department, Techno Main Salt Lake, Kolkata; Bolts- Comparing Capacities By Manas Mohon Ghosh, Consultant INSDAG; Durability Aspects of The Cold-Formed Steel Structures by V. Marimuthu, P. Prabha, M. Saravanan and M. Surendran, SERC,Chennai.

We believe that the range and scope covered by the technical papers in the yearbook covering the high strength steels, new steel materials like welded wire mesh, parallel flange sections, composite construction, corrosion protection of steel structures and innovative use of steel hollow sections will definitely create interest in steel fraternity and increased use of steel intensive structures.

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COMPARATIVE ANALYSIS AND OPTIMIZATION OF STEEL IN PRE-ENGINEERED BUILDING

Prof. A.J. Shah, Associate Professor,

Structural Engineering, DoCE, SVNIT, Surat

Kalpesh Jeengar, PG Scholar,

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ABSTRACT

Pre-Engineered Building systems are the buildings predesigned as per the requirement and prefabricated off the site as per the specifications and then transported to the site for the erection process. The current building strategies and requirements necessitate the best architectural appearance, high quality and Speed of construction, cost-effectiveness, and creative touch. For such requirements, some alternative construction technologies, such as pre-engineered steel structures, must be considered. The use of a Pre Engineered Building (P.E.B.) is a new idea that involves employing a steel framework and maximizing the Design while preserving economic integrity. Compared to other developed construction strategies, Pre Engineered Buildings are more sustainable and take first place. If we use ordinary steel structures, the time required from design to erection will be longer, and the cost will be more than if we use Pre Engineering Building. The materials employed in this idea are reusable, recyclable, and environmentally beneficial.

The present study's major goal is to compare pre-engineered steel structures with ordinary steel buildings in every way. A model of Pre Engineered Building was created, and a comparative analysis with a conventional steel building of the same dimensions and specifications was performed.

Keywords: P.E.B.(Pre-Engineered Building), C.S.B. (Conventional Steel Building), Pre-fabricated, Utilization Ratio.

1. INTRODUCTION

Steel construction use has expanded dramatically in the last two to three decades. As the earthquake forces significantly depends on the weight of building, steel buildings being lighter in weight than concrete buildings, steel buildings are more earthquake-resistant than concrete ones, still the concrete buildings are still preferred. As we know as soon as an earthquake occurs, its effects started to show much earlier in the form of cracks or spalling of concrete in RCC buildings as compared to steel buildings. Steel buildings come in various styles that are classified based on the community's needs. The pre-engineered steel structure is not a recently implied idea in the worldwide steel construction business. Still, their use in India is confined to a few accessible places. Pre-engineered structures are the most advanced type of steel buildings and are widely preferred in this regard.

Until 1990, pre-engineered structures were primarily used in the Middle East and North America. The utilization of pre-engineered structures has expanded throughout Africa and Asia, where the P.E.S. building concept is generally recognized and applauded. This expansion of the use of pre-engineered structures has occurred in recent years. Recently, a growing number of major global contractors and designers, who had previously only specified conventional structural steel structures, have begun to use the Pre-Engineered Building technique. As a direct result of implementing this strategy, they have been able to realize sizeable cost reductions and a shortened construction timeline. There is no other construction method that can compete with the Pre-Engineered Building system in terms of Speed and Value, beginning with the excavation and continuing all the way through occupancy. The extraordinary growth experienced by the pre-engineered steel (P.E.S.) industry over the course of the past half-century can primarily be attributed to the numerous advantages offered by pre-engineered steel structures (P.E.S.).

Small automobile parking shelters, wide clear span airplan hangars to low-rise multi-story structures are all possible applications. The pre-engineered structural concept has been used to achieve almost every possible architectural application. The demand for the use of pre-engineered structures is steadily expanding. It is due to the benefits it has over all other forms of steel construction. This form of the building is not only cost-effective but also environmentally beneficial. Because the steel industry has no negative environmental effects, steel building construction is more sustainable than other types of construction. The history of Pre-Engineered Buildings reveals that, until the last two decades, the use of pre-engineered structures was restricted to western countries alone (such as Britain and America), but with time and efficiency, it has spread to practically every country on the planet.

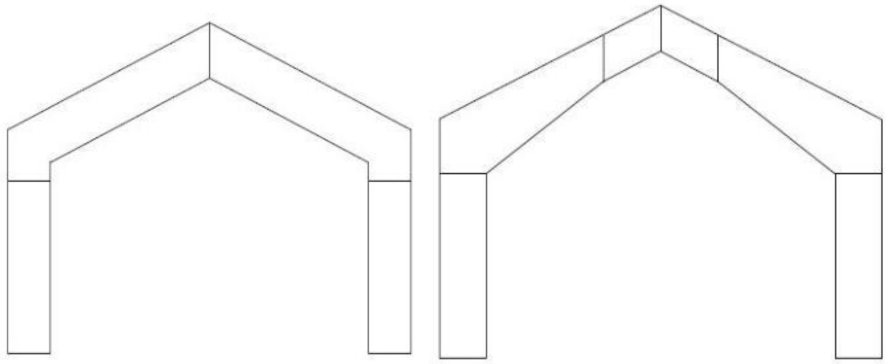

Figure 1 Conventional Steel building frame and Pre Engineered building frame, respectively

Pre-engineered structures are made up of a variety of structural and nonstructural parts that vary based on their form, size, construction, and Design. Columns, beams, rafters, purlins, bracing, sag rods, and girts are all examples of structural elements. Columns are the vertical parts of pre-engineered structures, whereas rafters are the horizontal members. The columns and rafters are the major structural parts of a pre-engineered structure. Other members of Pre-Engineered Buildings, such as roof purlins, girts, and sag rods, are referred to as secondary members.

2. TAPERING OF SECTIONS

From the basic knowledge of the load resisting mechanism of a steel section, it is known that the flanges of the I-section are effective in resisting the bending forces while the web of the I-section is effective in shear forces. As far as axial forces are concerned steel is assumed to be equally effective against axial compression and tension.

So the sections which are to be fabricated, instead of using the Parallel Flange section, we can provide a prefabricated tapered section as per the Bending moment and Shear Forces diagram of the member. The tapering of the sections is done in this project is as follows:

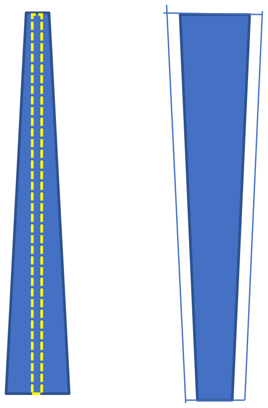

Figure 2: Tapering of Flanges and Web in the columns of the structure respectively.

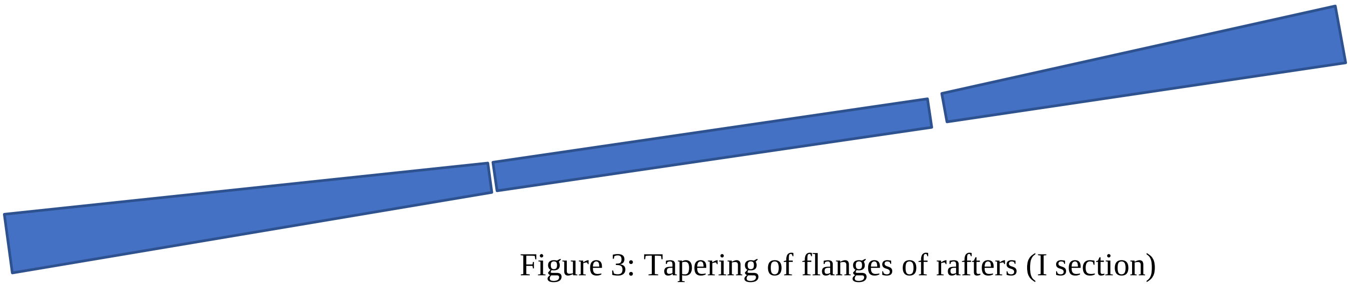

Figure 3: Tapering of flanges of rafters (I section)

Problem Statement

The plan area of the building is 45m x 80m with 7.5m bay spacing in the X-direction and 5m bay spacing in Z-direction. All the building models are designed to resist gravity loading and wind loadings. Seismic load is not considered in the Design as the objective of this study is to evaluate the minimum level of protection against wind loads available to a building. In a case where seismic load governs the design, the structure is expected to have a higher level of Wind Load protection. The buildings are analyzed by code-based linear dynamic procedure, i.e., wind load analysis method given in I.S. 875:2015, and then designed according to IS 800: 2007 and IS 875 Part 1, 2, 3.

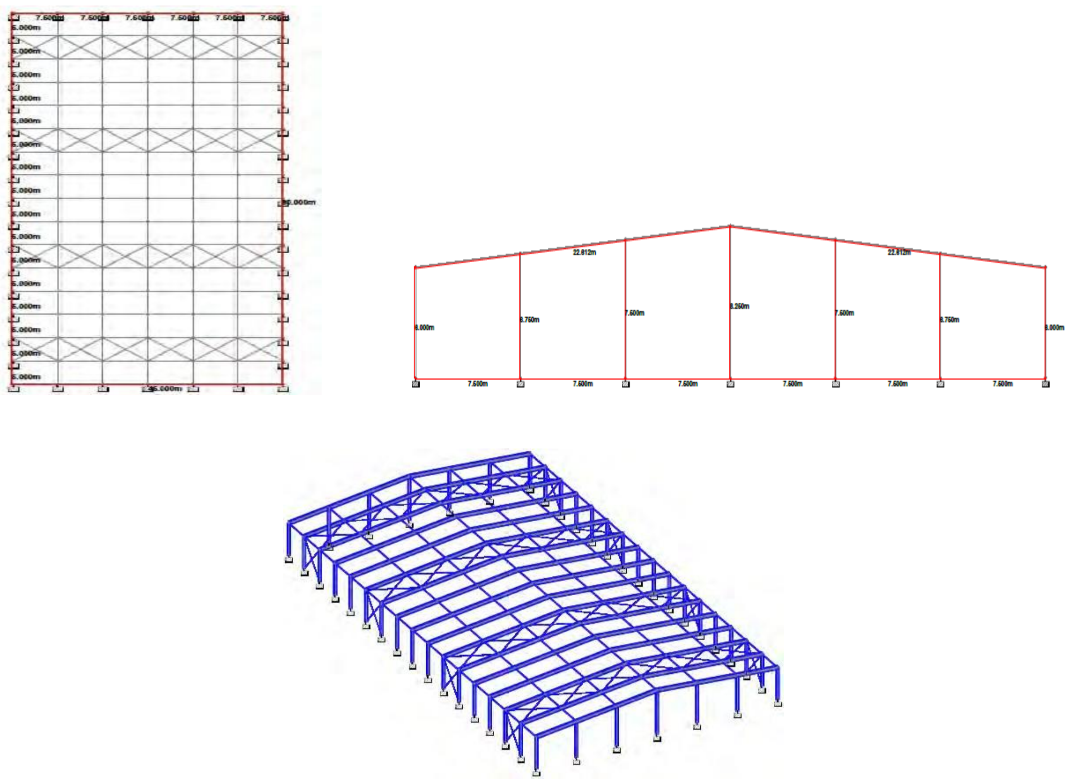

Figure 3 Plan, Elevation & 3-D View of the Building

The basic information related to the structure, such as geometric details, loading details, and preliminary dimensions, is shown in the below tables.

Table 1 Basic Information of building.

Building type and location	The industrial structure in Surat, Gujarat
Plan dimensions	45m×80m
Building total height	8.260 m
Typical story height up to eaves	6m
Slope of the roof	1:10 (5.71˚)

Table 2 Loading Data

Dead load	
Self-weight of all the steel section	To be computed by the software
Self-weight of claddings (Class A, 1.6mm thick)	0.131 KN/m2
Live load	
Live load on roof (inaccessible)	0.75 KN/m2
Wind Load	
Basic Wind Speed	44 m/s
Risk factor	1.0
the terrain height and structure size factor	0.91
the topography (ground contours) factor	1.0
Cyclonic Region factor	1.15
Directionality factor	0.9
Area averaging factor	0.9
Combination factor	0.9

The preliminary section sizes were obtained by doing approximate calculations of gravity loads. Then using the section database of the software Staad. Pro, the sections were revised till they satisfied the criteria of I.S. 800:2007.

Design Calculation

Design Wind Speed	**Vz = Vb X K1 X K2 X K3 X K4**				
VZ =	**Vb**	**K1**	**K2**	**K3**	**K4**
VZ =	**44.00**	**1.00**	**0.91**	**1.0**	**1.15**
VZ =	**47.00**	**M\S**			
Design Wind Pressure	**Pz=0.6×Vz^2**				
Pz=	**0.60**	**Vz^2**			
Pz=	**1325.40**	**N/m^2**			
Pz=	**1.33**	**kn/m^2**			
Note:	**The Value of Pd, However, shall not be taken as less than 0.70 pz**				
0.70Pz=	**0.93**	**KN/m^2**			
design wind pressure	**Kd × Ka × Kc × Pz**				
Pd =	**Kd**	**Ka**	**Kc**	**Pz**	
Pd =	**0.90**	**0.90**	**0.90**	**1.33**	
Pd =	**0.97**	**Kn/m^2**			
Wind Load on Individual Members	**F=(Cpe - Cpi) ×A × Pd**				
		90 Degree (A)	**0 Degree (A)**	**Pd**	
		6.67	**5.00**	**0.97**	
IS -875 Part-3					
Cpe (External pressure coefficient)					
	A	**B**	**C**	**D**	
Cpe (0) Degree	**0.70**	**-0.20**	**-0.50**	**-0.50**	
Cpe (90) Degree	**-0.50**	**-0.50**	**0.70**	**-0.20**	

Cpi (internal pressure coefficient)					
Plus Cpi	**0.5**	Cpi for 5% -20% Opening			
Minus Cpi	**-0.5**				

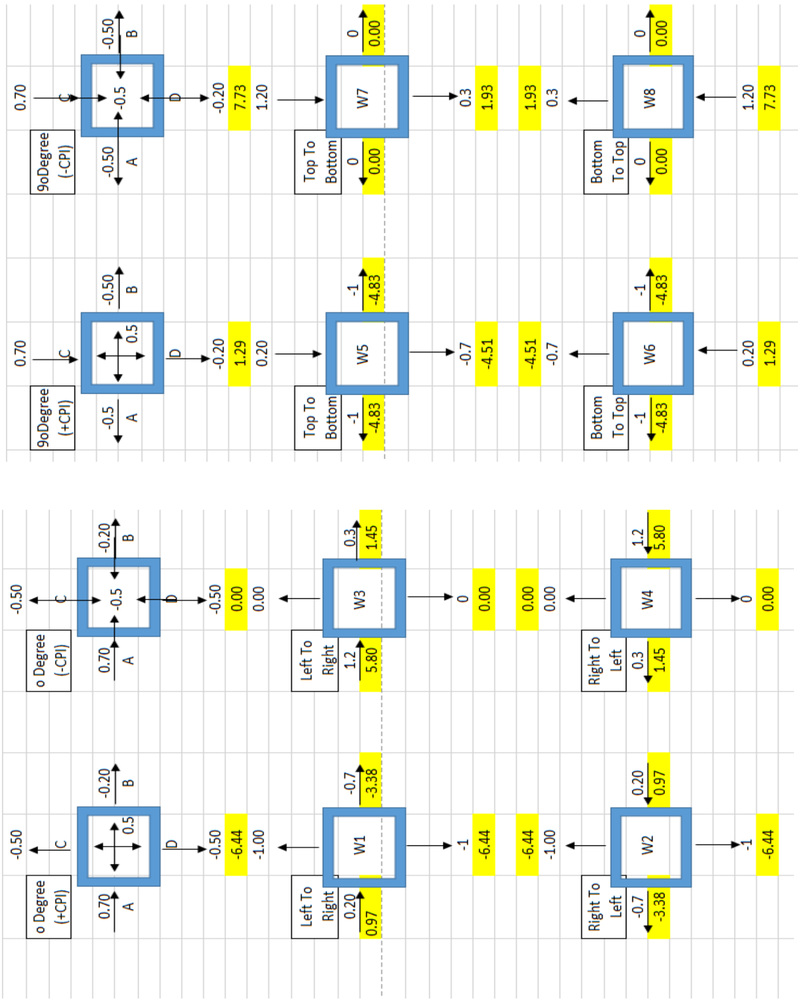

Figure 4 Wind load calculation with different cases

Staad.Pro Modelling and Analysis

Now let us have a glance at what STAAD. Pro Connect Edition software look like and also look at its features and what methodology is used to analyze the building.

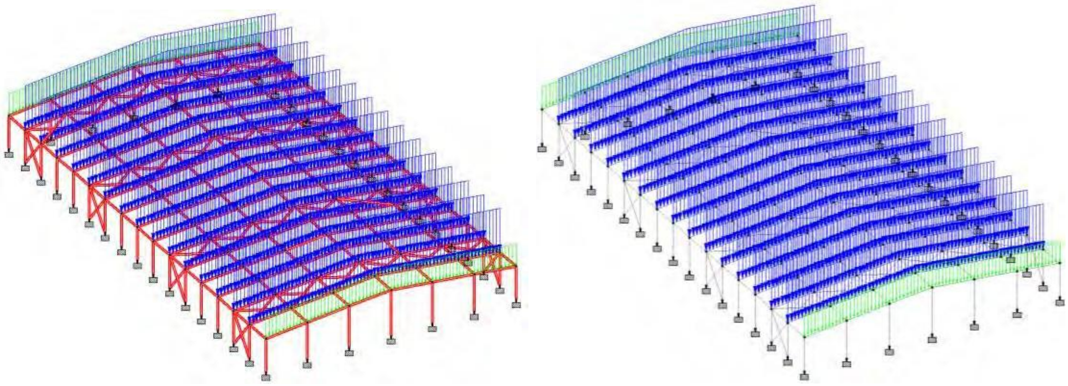

Figure 5 Dead &Loads applied to the structure

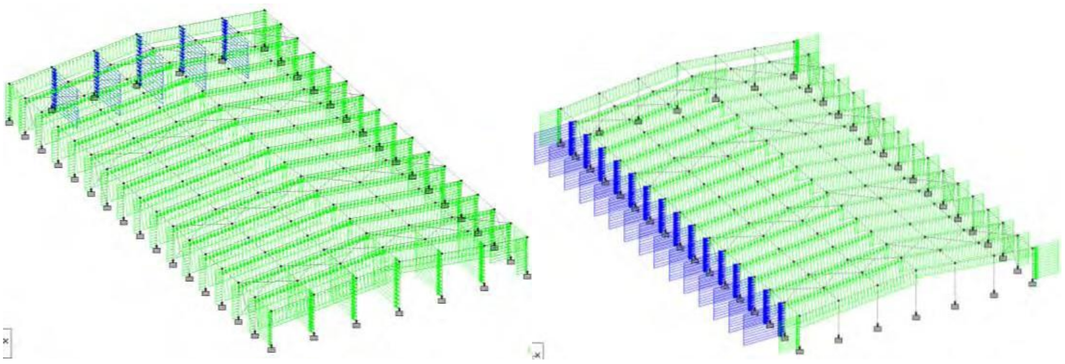

Figure 6 Wind Loads in +X direction with +Cpi & -Cpi

Similarly, Wind forces were applied from +X direction with +Cpi, +X direction with -Cpi, -X direction with +Cpi, -X direction with -Cpi, +Z direction with +Cpi, +Z direction with -Cpi, -Z direction with +Cpi, -Z direction with -Cpi. The Wind loads were transferred from cladding to the columns and rafters. Hence, in software, the cladding is not defined, and the loads were directly applied to the columns and Rafters.

3. RESULTS AND DISCUSSION

After designing the building by a conventional design approach and a Pre-Engineered design approach, the structure needs to be assessed for its satisfactory performance against different loading and load combinations. Being a low-rise steel structure and light in weight, the seismic forces were not being considered for such type of structure; wind loads are the critical loads. The main aim of Pre-Engineered Design is to optimize the steel quantity being used for the construction and increase the utilization ratio of the section as compared to the Conventional steel design for a long-span column-free area as needed for an industrial shed.

There are several aspects with respect to which the comparative study can be made:

1. Steel quantity used for the structure
2. The utilization ratio of the individual structural elements.
3. Lateral stability against lateral loads.
4. Foundation Requirements

4. STEEL QUANTITY USED WHOLE STRUCTURE

1. For Pre-Engineered Building

PROFILE	LENGTH (METER)	WEIGHT (KN)
Tapered Member 1	536.84	275.154
Tapered Member 2	256.27	127.412
Tapered Member 3	256.27	123.475
ISMC 70*70*8	560.33	18.620
ISMC100*100*10	560.00	52.606
Total		**597.267**

2. For Conventional Building

PROFILE	LENGTH (METER)	WEIGHT (KN)
ISHB450H	277.50	249.414
ISHB300	768.82	441.769
ISMC100*100*10	560.00	52.483
ISMC 70*70*8	559.13	18.256
Total		**814.528**

Figure 7 Total Steel Required for P.E.B. and C.S.B.

Looking at the above data, it can be concluded that the weight of a Pre-Engineered Building is about **21%** lighter than the Conventional Steel building, and it can further be reduced by hit and trial and checking the structure on every sectional variation according to bending moment values at a different location in.

5. UTILIZATION RATIO OF THE INDIVIDUAL STRUCTURAL ELEMENTS.

	Conventional Steel Building		Pre-Engineered Steel Building	
	Column	Rafters	Column	Rafters
Max Utilization	86.5 %	56.5 %	96.3 %	72.8 %
Min Utilization	26.2%	30 %	67.3 %	66.2 %

As we can see from the utilization ratio of columns and rafters of the conventional structure, the ratio of most of the columns is less than 30%, and for rafters, it is less than 60%, which means the section is not being stressed up to 30% and 60% of their total capacity respectively. Due to this, most of the steel sections remain unused, and the structure becomes uneconomical if the plan area is large.

While if we see the utilization ratio of the columns and raters in the case of Pre-Engineered Buildings, the columns are being utilized from 67%-98%, and the rafters are being utilized up to approximately 70%. This utilization ratio in the case of Pre-Engineered Buildings can be further increased beyond 90% by hit and trial procedure. This means the sectional efficiency of the Pre-Engineered Building is so far better than the Conventional Building.

Hence, we can say that in Pre-Engineered Buildings, the sections are being used more efficiently (more than 70%) as compared to conventional steel buildings, which makes the Pre-Engineered Building a more economical and engineered approach for the Design of a structure.

6. LATERAL STABILITY AGAINST LATERAL LOADS.

Deflection of Conventional Steel building and Pre-Engineered Steel building under lateral loads.

	Conventional Steel Building	Pre-Engineered Steel Building
Maximum Deflection	0.93 mm	9.31 mm

As we can see, In the case of Conventional steel buildings, the lateral deflection is much less than in Pre-Engineered Buildings, which means the Conventional Building design is much stiff, which is not desirable as far as lateral loads (wind and earthquake loads) are considered. As if lateral loads exceed the capacity of the structure, the structure being stiff in nature will fail as the brittle failure of the structure as a whole and the ductile property of the Steel could not be used.

But, in the case of Pre-Engineered buildings, the structure is more flexible and light in weight. The flexibility of the structure enables ductility to come into action when lateral loads are applied. And being light in weight makes it less prone to earthquake loads. Hence, in Pre-Engineered Buildings, the wind loads are more critical loads than earthquake loads as compared to Conventional Steel buildings.

7. FOUNDATION REQUIREMENTS

The type of foundation to be provided for a particular structure depends on the load-bearing capacity of the underlying soil and load applied or the reactions at the bottom of the structure. The following table shows the maximum and minimum reaction in X,Y & Z direction at the base of Conventional and Pre-Engineered buildings.

Reaction	Conventional Steel Building	Pre-Engineered Steel Building
Max Fx (KN)	105.4	81.3
Min Fx (KN)	140.6	103.6
Max Fy (KN)	47.5	31.8
Min Fy (KN)	8.7	6.1
Max Mz (KN/m)	48.6	30.3
Min Mz (KN/m)	4.2	1.3

Looking at the reaction provided at the base of the Conventional Steel building as well as the Pre Engineered Building. We can say that the reaction in the X and Z directions does not contribute to the type or size of the foundation, and these forces are resisted by the base plate with the help of J-bolts and cleats. At the same time, the moments in the X and Y directions are negligible in most of the columns. Looking at the reactions in the Y-direction and Moment in Z-direction, which are the deciding factors for the foundation, we can see the reaction in the case of the C.S.B. structure is higher than the P.E.B. structure.

This means the Conventional Steel building (C.S.B.) requires a heavier foundation as compared to the Pre-Engineered Building. This makes the CSB less economical than PEB as the extra cost of the foundation has incurred.

8. CONCLUSIONS

This project is chosen in order to compare the two approaches, which are the conventional steel design method and the Pre-Engineered Building design method, and to arrive at the conclusion of choosing Pre-engineered Design over the conventional design method for designing steel structure. Based on the analysis, design, and performance assessment of buildings designed by conventional approach and newly emerging Pre-engineered design approach, the following concluding remarks were made.

- Low cost, strength, durability, design flexibility, adaptability, and recyclability are some of the benefits that pre-engineered steel structures offer in a building. Steel is the primary component of all of the other materials that are utilized in the construction of pre-engineered steel buildings. It contradicts evidence from local sources. Steel, which can be recycled indefinitely, is the material that most accurately reflects the requirements of sustainable development.

- As it can be seen in the present work for the designed Industrial structure, the weight of the Pre-Engineered Building was almost 20% less than the Conventional structure, which can be further reduced depending on the manufacturing capability of the laboratories and the designer.
- Looking at the utilization ratio of the section in the case of Pre-Engineering Building structure which is much greater than C.S.B., which means the section is being used at its greater potential, and the usage of the Tapered section made it possible to reduce the steel quantity used for the same structure as Conventional steel structure.
- Conventional buildings are not appropriate for long-span structures because they do not have clear spans. As can be seen in this current work, an industrial structure has been designed for 45 meters, and Pre-Engineered Buildings are the optimal solution for long-span structures that do not include any interior columns in between the spans. Because of the rise of computer technology, the potential for Design has expanded to an almost unfathomable degree.
- Prefabricated buildings are more cost-effective than conventional steel buildings due to the reduced amount of material used in low-stress areas of the primary framing members. This is especially true for low-rise buildings that span up to 45 meters and have eave heights of up to 5 meters. When compared to conventional structures, Pre Engineering buildings tend to have a higher price tag, particularly in the case of structures with a shorter span.

In conclusion, "Pre-Engineered Building Construction gives the end-users a much more economical and better solution for long-span structures where large column-free areas are needed."

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EFFICIENCY OF STEEL DIAGRID BUILDINGS WITH BUCKLING-RESTRAINED-BRACES UNDER EARTHQUAKE LOAD

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ABSTRACT

The word 'Diagrid',a combination of 'Diagonal' and 'Grid', is gaining popularity among structural engineers now-a-days due to its aesthetic appearance. Diagrid structures are comprised of mainly diagonal grids which are basically bracing members. The lateral load flows through the system by the axial action of the bracing members. The brace members are generally weak against compression due to the buckling of the brace. So, the Buckling-Restrained Brace (BRB) is felt to be useful to counter that phenomenon of buckling of conventional steel braces. This study presents the comparative assessment of seismic parameters of Conventional buildings and Diagrid buildings with conventional brace and BRB.

Keywords: Buckling-Restrained Braces (BRB), Diagrid Structures, Concentric Bracing system, Non-linear static pushover analysis, Time-history analysis.

1. INTRODUCTION

The rapid growth of the urban population, the lack of space in the cities, and the high cost of building materials impact the designers to focus more on buildings vertically rather than horizontally. And as the height of the building is increasing, the lateral load resisting system becomes more important than the gravity load resisting system(Trica and Gioncu, 1999), (Pattar and Gokak. 2018). Among all the popular structural systems used for high-rise buildings, Diagrid structural system is becoming more attractive in recent years due to the aesthetically appealing look provided by the unique geometric configuration of the system (Sabuz et. al, 2021). A diagrid basically consists of diagonal bracing system, which constitutes the main load transferring mechanism (Fig. 1).However, it is well recognized that, bracings may easily fail in compression due to buckling, and render the structure to perform inefficiently during earthquakes(Moon, et. al. 2007). With an end-in-view to improve the seismic performance of such bracings, a new concept of Buckling restraint braces (BRBs), instead of conventional braces, is getting popular in the last decade for ordinary braced moment resisting frames (Deulkar et al., 2010; Takeuchi and Wada, 2018). Normal bracings are not capable of yielding in compression due to buckling. So, using BRBs in place of normal bracings in diagrid will allow one to get the yielding of the bracing members both in tension as well as in compression(Xie, 2005). Thereby, the strength of steel braced frame with BRB will obviously increase by eliminating the chance of buckling to a maximum extent (Guerrero and Escobar,2016). Sy et al. (2014) described the application of BRB in tall structure for design based earthquakes using response spectrum analysis. For credible earthquakes, the authors used non-linear time history analysis. The outcome show that the BRB is effective in lowering base shear and regualating deformations (Pan et. al. 2019). This led us to adopt BRB in diagrid system as well.The BRB consists of a steel brace in the core and a concrete or mortar like material restraining the buckling of the steel core encased by a steel tube (Fig. 2). The concrete or mortar acts like an elastic support to resist buckling of the steel bracing. A debonding material is used between the steel core and the concrete so that the steel brace can slide within the restraining concrete-filled steel tube with ease, and no frictional load will act on the BRB core that induce extra compressive or tensile stresses. Research is going on to amend the conventional version of BRB to further improve its seismic performance by hybrid BRB (Das and Deb, 2022), where a detachable BRB core is joined along the length of ordinary steel core to facilitate monitoring of seismic retrofitting. Also, Heshmati et al. (2022) proposed another hybrid BRB, where two types of steel, one with low yield point and another with high performance characteristics are combined to form the BRB to make the structure seismic resilient. However, the conventional BRB, already mentioned in the former part of this paragraph has already achieved desired milestone through practical applications (Fig. 1) and hence adopted in the presented study.

The research on seismic performance analysis of diagrid systems are also gaining increasing attentionin the last decade. Jani and Patel, (2013)presented analysis and design of diagrid structural system for high rise structureswith a 36-storey diagrid steel structure.Shah et. al, (2016) compared diagrid structures with conventional frame structures under lateral load with a 24-storeyed steel buildings. Sadeghi and Rofooei (2020) studied seismic performance of diagrid structures. The authors have taken three 8-storeyed and three 12-storeyed diagrid structures with different diagrid angles, and observed the optimal angle. The diagrid system adopted in this study is based on the observation of Sadeghi and Rofooei

(2020).Dabbaghchian (2021) reported inefficiency of ordinary diagrid system under seismic effects. The authors proposed shear fuse and eccentric bracing system to improve the seismic efficiency. However, in the present study, the diagrid with concentric bracing system is used as such system has already constructed in important projects (Fig. 1) and needs no practical validation. Only, instead of conventional braces, BRB is used in the bracing system to visualize the improvement achieved by the proposed BRB-equipped diagrid system. Since, not much study is observed in this field, the present paper seems to be a useful contribution. Sadeghi and Rofooei (2020) is the only available literature, where BRB-equipped diagrid system has been attempted, but the authors aopted the approach of Malley (2007), which is as per AISC specifications. Also, the authors adopted pushover analysis and non-linear time-history analysis approach for seismic evaluation of BRB-equipped diagrids. It is thus felt required to explore this BRB-equipped diagrid system as the proposed system with Indian code of practices IS 1893(2016); IS 875 (1) (1987); IS 875 (2) (1987), IS 875 (3) (2015) and IS 800 (2007). Also, since seismic coefficient method and response spectrum analysis as per IS 1893 (2016) is the current seismic analysis appraoch in India, assessment of BRB-equipped diagrid with these appraoches seems to be of more practical relevance for Indian steel construction industry. Thus, the present study marks a unique contribution in this field by exploring effectiveness of proposed BRB-equipped diagrid system using Indian codal provisions and the sesimic analysis appraoches mostly adotped in India. In the international perspective, this paper will be the second study to explore effectivenss of BRB-equipped diagrids after Sadeghi and Rofooei (2020), though, as mentioned, the approach adopted in the present paper is distinctly different from Sadeghi and Rofooei (2020), both in practical relevance, application examples, and methodology.

(b) The Doha tower, Qatar

*Ref:*https://images.app.goo.gl/SMZnMBiSwjJeapwv7

(c) The Swiss Re building, London

*Ref:*https://images.app.goo.gl/NRx6T4oZ2QVK6PPPA

(d) The Tornado Tower, Qatar

*Ref:*https://images.app.goo.gl/m48SDHfTLHP1qwtU8

Fig.1 Practical application of diagrids

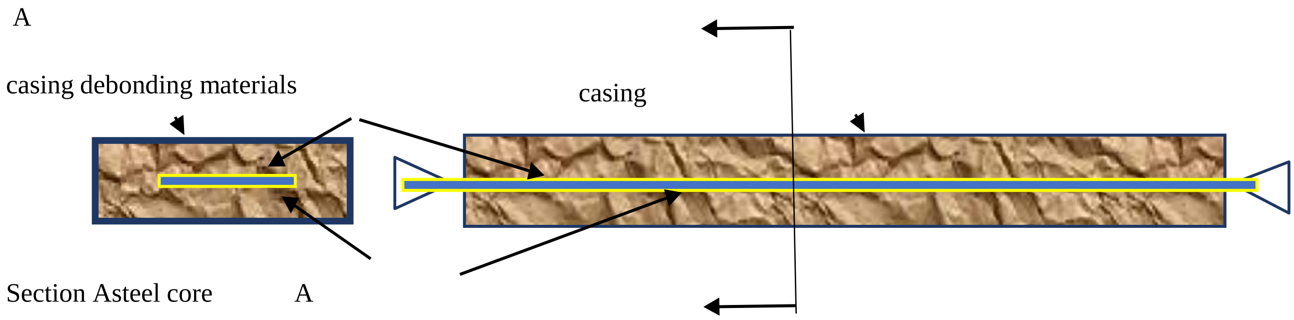


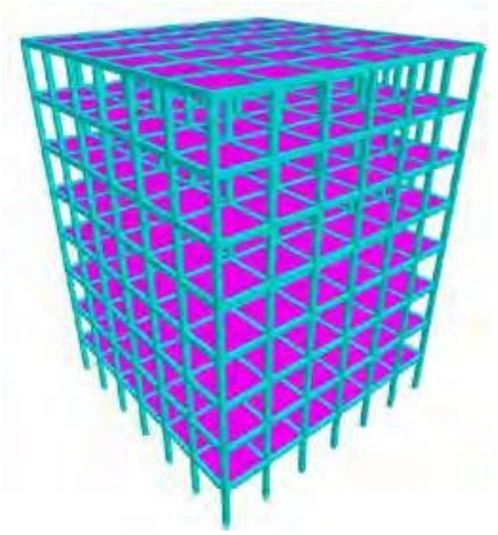
Fig. 2Cross-section and longitudinal section of BRB

2. DIAGRIDS OVER CONVENTIONAL FRAMED STRUCTURE

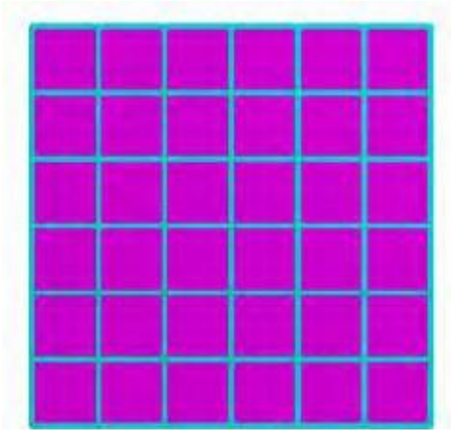
In order to visualize how the diagrid structures offers better seismic load resisting systems than conventional moment resisting frame systems, an eight-storeyed building has been taken up. The dimensions of the frame elements and building models are shown in table 1 and Fig. 3, respectively. STAAD.Pro software is used to model the structures.

Table 1. Dimensions of structural elements

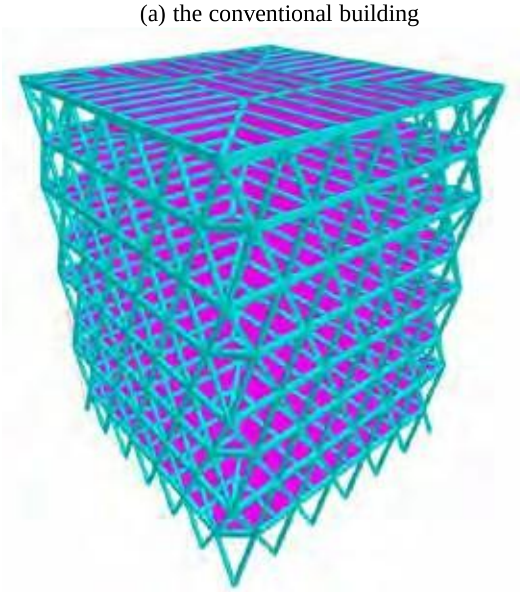
	Conventional Building	Diagrid Building
Beam	350 mm×230 mm	350 mm×230 mm
Vertical columns	300 mm×300 mm	300 mm×300 mm
Diagrid columns	Nil	B 200×17 mm
Slab thickness	150mm	150mm



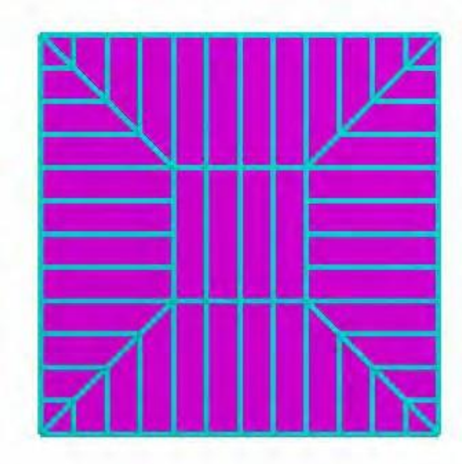
(a) the conventional building



(b) Beam layout of the conventional building;



(c) the diagrid building



(d) Beam layout of the diagrid building

Fig. 3. The eight-storied building

It may be noted that the beam layout of diagrids is different from the beam layout of the conventional building as diagrid doesn't have any vertical columns in the external periphery. Rather, the vertical columns are situated in the internal core of the building only. The diagonal bracings present in the diagrid building are connected with ring beams and these ring beams are connected with the core of the building with the help of tie beams.

The loading conditions that the buildings are subjected to are: i) dead loadin terms of the self weight, floor weight of 4 kN/m^2 and an UDL of 15 kN/m for outer wall and 7.5 kN/m for inner wall over the beams; ii) live Load applied of 3 kN/m^2 over each floor and 1.5 kN/m^2 over roof. iii) earthquake load, considering seismic zone V, importance factor 1.2, response reduction factor 4 as per IS 1893-2016. The response spectrum method is used. iv) wind load considering basic wind speed of 50 m/s as per IS:875(III)-2015.

Theresults are shown in the Table 2 in terms of maximum moment, maximum shear, maximum axial force induced in the structure, maximum deflection, and storey drift for both the conventional and the diagrid buildings. The better response quantities are highlighted in bold-faced fonts. It can be observed that except for the torsional moment (which is itself marginally small), the diagrid system yields better response quantities for all other response quantities. The storey drift and deflection is substantially reduced for the diagrid system compared to conventional framing system. The less base shear by the diagrid systems indicates that the diagrid system consists of lesser structural weight than the conventional framing system.

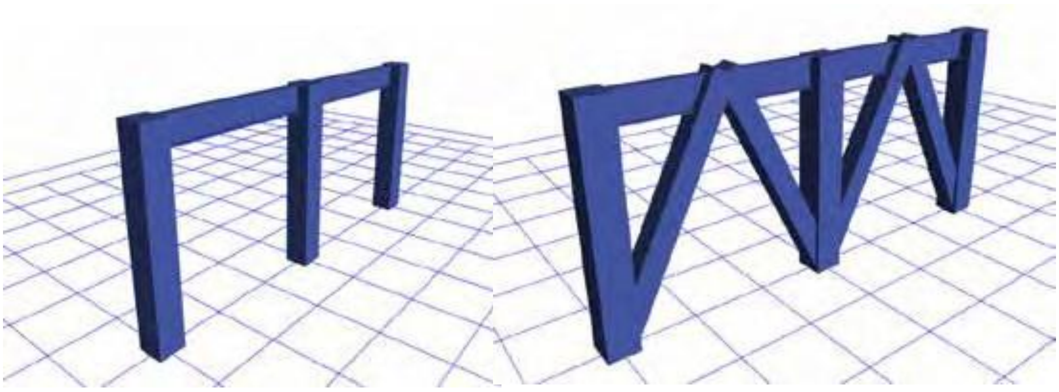
Table 2. Comparison of response quantities for conventional buildings and diagrids

Response quantity	Conventional framing	Diagrid
Maximum torsional moment (kN-m)	**4.9**	7.6
Maximumbending moment (kN-m)	164.9	**47.1**
Maximumshear (kN)	138.3	**72.5**
Maximum axial force (kN)	3839.1	**1377.0**
Maximumdeflection (mm)	88.1	**9.3**
Base shear (kN)	1883.1	**1261.3**
Maximum storey drift (mm)	16.6	**1.4**

3. BRB FRAME OVER CONVENTIONAL BRACED FRAME

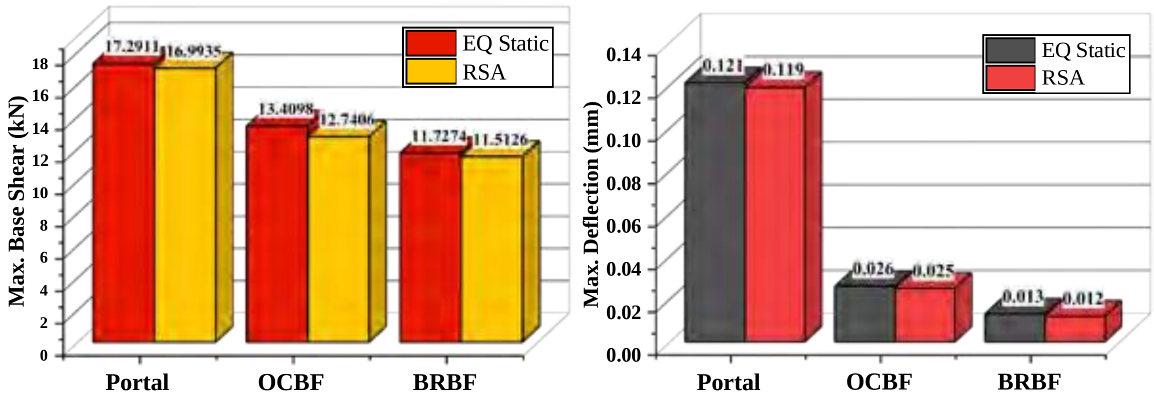
The frameswith BRB are better in resisting the lateral loads when compared to Ordinary concentrically braced frames (OCBF) because of their ability to yield in the compression as well (Sadegi and Rofooei, 2020). This example problem is taken to further quantify the advantage one can get from a BRB-equipped frame. A single storey two-bay frame has been modelled with BRB as well as conventional bracing system. A simple portal frame (without bracing) has been also considered. The beam sections are taken as ISMB 400, the column is of B 400×45. The concentrically braced frame has bracing of B 200×10, which is obtained after designing the frame most economically. Similarly, StarBRB_30.0 section available in ETABS is used for BRB-equipped frame to yield most economical design. The StarBRB_30.0 section comprises of 300 mm × 300 mm outer casing with inner core area

193.5 cm^2.The length of yielding core is 2.12 m. The three dimensional view of portal frame and braced frame is shown in Figs. 4 (a), and (b), respectively.

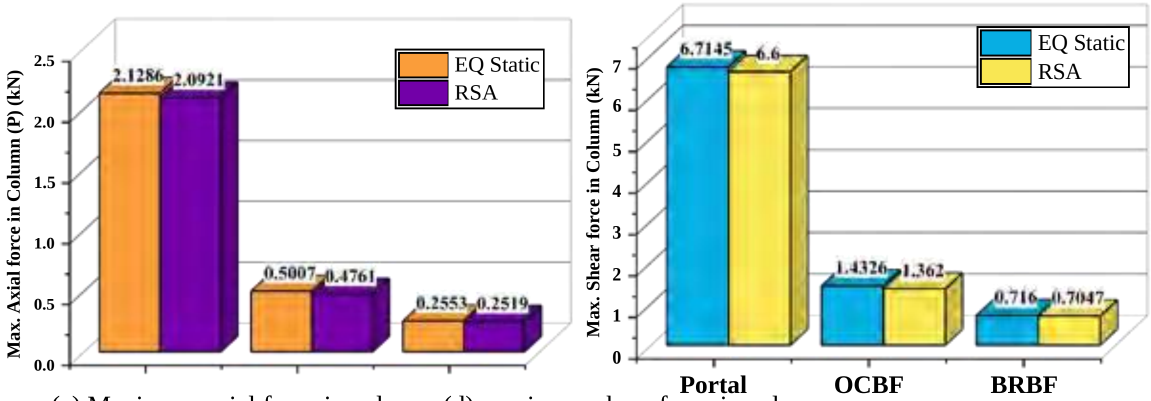


(a) Portal frame (b) braced frame
Fig. 4: three-dimensional view of the frame

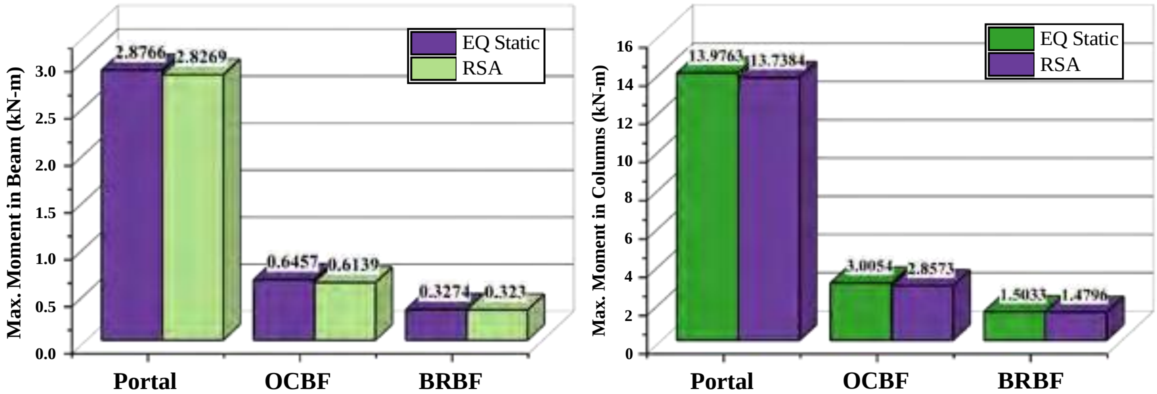
The frame is analysed for self-weight of the structure, and 15 kN/m uniformly distributed load over the beam. Like the previous problem, both the response spectrum analysis (RSA) as well as equivalent static method (EQ Static) are adopted to estimate the seismic force. The base shear, maximum lateral deflection, maximum axial force in column, maximum shear force in column, maximum moment in beam, and column are plotted for all the three framing systems in figures 5(a)-(f), respectively. It can be observed that in all cases BRB Frame (BRBF) yields better response than the OCBF. The response quantities observed for ordinary portal frame is the worst among the three system.



(a) Base shear (b) Maximum Lateral deflection



(c) Maximum axial force in column (d) maximum shear force in columns



(e) Maximum moment in Beams(f) Maximum moment in Columns

Fig. 5. Comparison of seismic responses in the frame

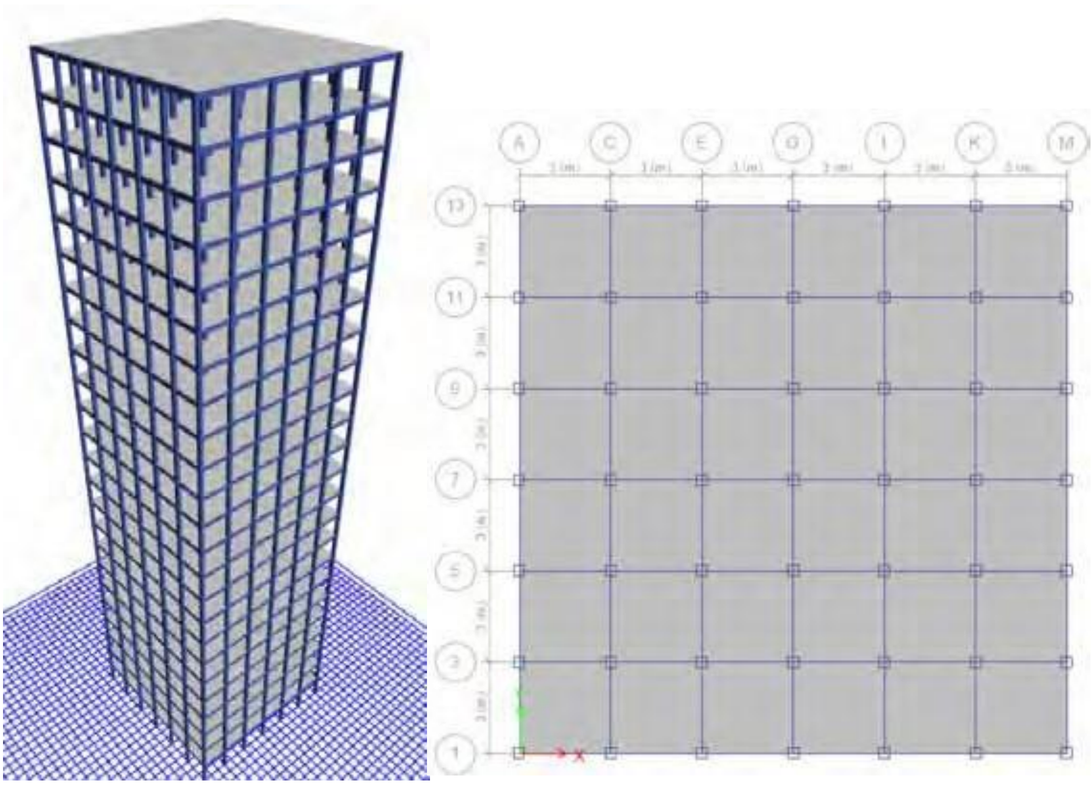
4. BRB EQUIPPED DIAGRID OVER CONVENTIONAL DIAGRID BUILDING

Already in the last two sections, the efficiency of the diagrids and BRB have been established with numerical examples. In this section, these two systems are combined to frame BRB-equipped diagrid buildings to utilize their individual capabilities of improved seismic resistance. This proposed system is compared with the conventional steel frame building, and ordinary diagrid building. For this purpose, a 24-storeyed building is taken up and modelled in finite element analysis software ETABS. The sections properties of various structural elements are presented in Table 3. The configuration of StarBRB_30.0 sections is already mentioned in section 3. For all the systems, the building is optimally designed to yield the section properties presented in Table 3.

A dead load of 15 kN/m is applied on the peripheral beams to simulate external wall loads.Dead load for internal wall is considered as 7.5 kN/m. For parapet wall 5 kN/m load is applied on the peripheral beams of the roof. A live load of 2.5 kN/m^{2}is applied over floors and 0.75 kN/m^2 is imposed over roof. As before, earthquake load is estimated for the same seismic condition as of previous examples, and structure is analysed by EQ Static, as well as RSA methods. The isometric view and typical floor plan of conventional frame are shown in Fig. 6(a) and 6(b), respectively. The same for diagrid system are shown in Figs. 7(a) and 7(b), respectively. In Fig. 7(b), plans of three consecutively floors are presented side-by-side. The beam layout plans remain same for the BRB-equipped diagrid building.

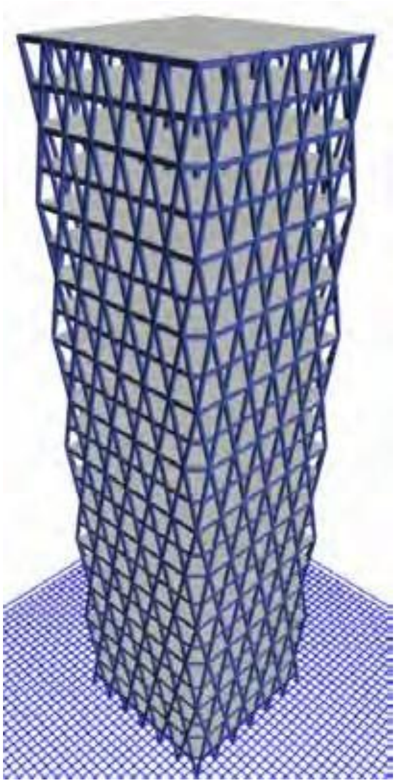
Table 3. Section properties of various structural elements of the 24-storied building

Structural Elements	Conventional Building	Normal Diagrid	BRB equipped Diagrid
Beam	ISMB400	ISMB450	ISMB450
Column	B- 350×35	B-400×15	B-400×15
Brace	NIL	B-200×10	StarBRB_30.0
Slab	Shell-thin, M25 concrete, 120mm thick		

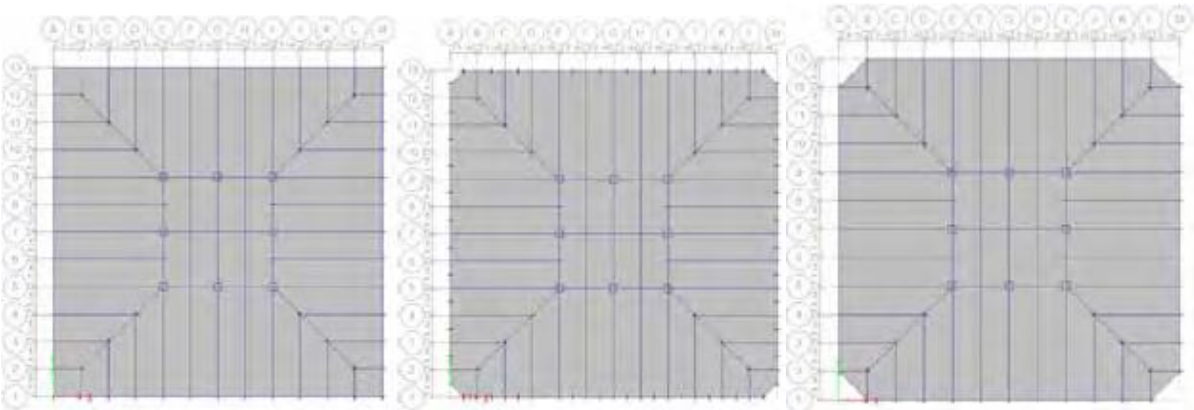


(a) Isometric view (b) beam layout plan

Fig. 6.24-storied building with conventional frame



(a) : Diagrid configuration of 24-storied building

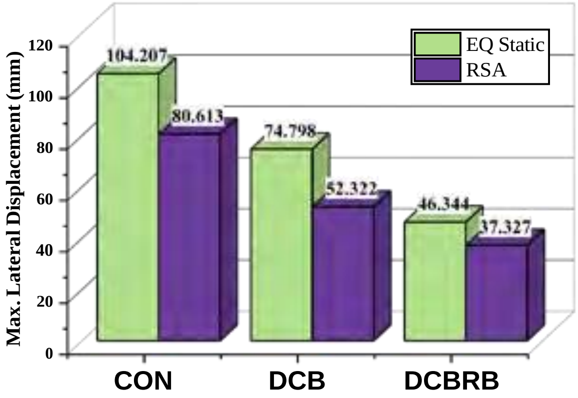


(b) Beam layouts of three consecutive floors of 24-storied diagrid building

Fig. 7: The 24-storied diagrid

The maximum lateral displacement and the maximum drift (as a ratio of inter-storey drift over the storey height) for the three systems: conventional building (CON), diagrid building with concentric bracing (DCB), and diagrid building with concentric BRB (DCBRB), are shown in Figs. 8 and 9, respectively. It can be observed that both the lateral displacement as well as the drift is minimum for the proposed DCBRB system, whereas, the CON system shows maximum displacement and drift. The observations remain same by both the EQ Static and RSA methods.

The maximum shear forces and the maximum bending moments among all the beams and columns, are shown in Figs. 10 and 11, respectively. The same observation of less bending moment and shear force for the DCBRB system is pertinent here also by both the seismic analysis approaches.



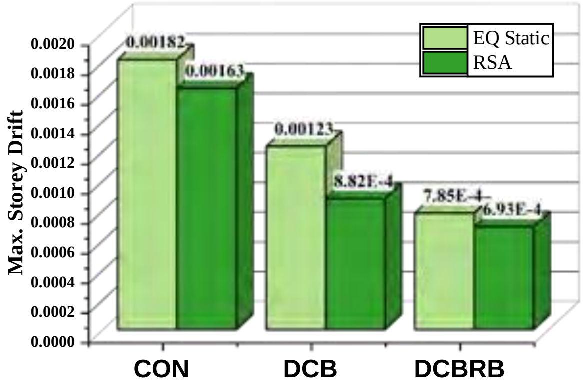
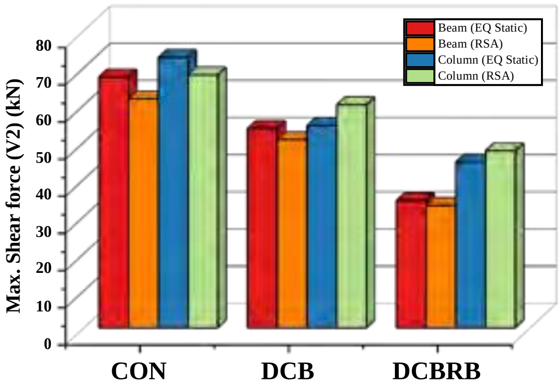


Fig.8The Maximum Lateral Displacement Fig.9 Maximum storey drift ratio



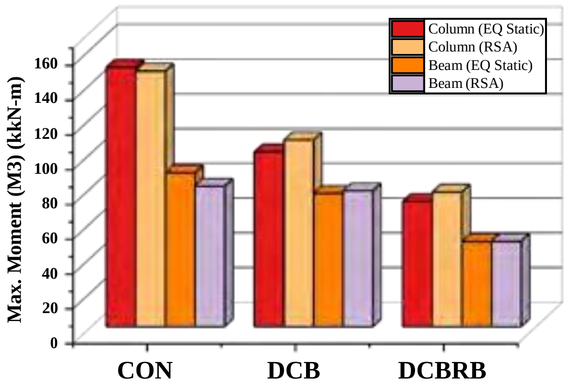


Fig. 10Maximum shear force in beams and columnsFig. 11. Maximummoment in members

The total weight of steel is compared in Fig. 12. The total steel weight is significantly lesser for DCBRB system compare to CON building. Surprisingly a 43% saving in steel is achieved. The DCB system also show 41% saving in steel weight. However, so far other seismic performance parameters are concerned (such as maximum lateral displacement, drift, maximum moment, maximum shear), the DCBRB system is already shown to be better than the DCB system. Hence, it will be quite relevant to explore and codify the design provisions of BRB equipped diagrids (DCBRB), because of its improved seismic performance and economy.

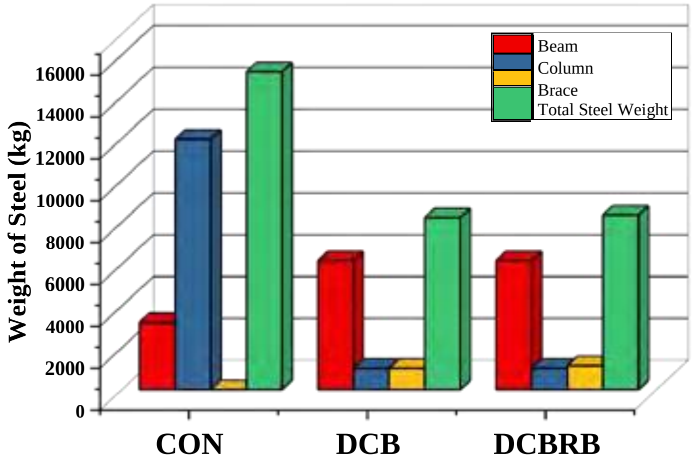


Fig. 12. Comparison of steel weight required by the three framing systems

5. CONCLUSION

In this study, seismic performance assessment of BRB equipped diagrid framing system is presented taking conventional steel moment resisting frame and ordinary braced diagrid systems as reference. Three numerical studies — i) a simple two-bay steel portal frame, ii) an eight-storied steel building, and iii) a 24-storied steel building, are executed to establish the potency of the BRB-equipped diagrid system. The results show that the proposed BRB-equipped diagrid system yields lesser displacement, drift, moment and shear in the building compare to conventional steel moment resisting frame and ordinary braced diagrid systems. The steel weight required by the proposed BRB-equipped diagrid is also less than the conventional ordinary braced steel framed building. These observations indicate the importance of laying codal stipulations for design of BRB-equipped diagrid buildings, and explore future possibilities of adopting such system in steel intensive building construction.

In the present work, EQ Static and RSA method of analysis is used for earthquake effect assessment, since these are most widely used approaches in industry. The finite element analysis software ETABS is used to model, analyse and design the buildings. As a future extension of this work, a pushover analysis, or time-history analysis may be executed. Exploring effectiveness of BRB-equipped diagrids for other irregular buildings in plane and sloped ground, and for taller building will be also relevant in this regard. Use of eccentric bracing instead of concentric bracing is also a topic worth exploring. Indeed, all these scopes will enrich the domain of steel development and growth for future era to come, where taller structure will become a most viable choice to cope with the problem of scarcity of land.

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BUILDINGS IN SEVERE EARTHQUAKE ZONES MADE OF STRUCTURAL STEEL HOLLOW AND PLATE MEMBERS

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ABSTRACT

In old days wooden buildings were made in highly earthquake prone areas for its low weight. Nowadays with the advancement of steel industry if we can make these type of buildings using square hollow sections/rectangular hollow sections, steel plated/wooden floors and puffed panel walling systems then it will be more strong and low weighted also. These types of buildings are green, sustainable and eco-friendly. In this paper one model building has been analysed for seismic zone IV or wind speed 47 m/sec as per Indian Standard. Purpose of the building is residential or normal office type. It is found that wind forces developed in the structural members are greater than the seismic forces generated in the same. Because of low mass seismic forces are not generated despite of heavy ground acceleration. Proper steel bracing systems are provided vertically and horizontally for stability of the structure. The production of new structural hollow members, chequered plates, puffed/sandwiched panels have created a new era in building industry making it more sustainable in all respects.

Key words: Severe earthquake zone IV, Steel building, Hollow structural members, Steel floor, Puffed panel walls, Low weight

1. INTRODUCTION

The purpose of study is to understand the effect of earthquake on such building in highly seismic prone areas and the goal is to find ways and means to control seismic effects on those buildings. It will encourage construction of such buildings in highly seismic areas if not in large scale, at least in the construction of building marked as important building which needs to provide service to the population immediately after the event (earthquake) or building which cannot afford to be dysfunctional, such as railway stations, airports, telephone exchanges, bureaucratic offices, police stations, army headquarters etc. for any period of time. Schools and colleges should also come under this category because effect of earthquake in such buildings as would be revealed from the study is limited or even if there is limited effect, swift restoration is possible in such buildings. This is primary aspect. This should be encouraged in all parts of the country irrespective of seismic zone. The other aspect is to encourage buildings higher than 4 storeys in hilly areas of zone IV and all building in hilly areas of zone V as per Indian Standard to be steel buildings. In plain areas of zone V discretion should be used by local authorities primarily keeping in mind the height and volume of the building, the inclination should be to encourage steel buildings using low weight partition and flooring as suggested in the study. The purpose through the study is to encourage low weight construction. Heavy RCC design method is not suitable in highly earthquake prone areas.

2. BACKGROUND

The paper is built on the well known back ground of the damage that is caused by earthquake both in terms of life and property, which are visible losses. But for me more than the visible losses are the invisible traumas that people face during the earthquake and a certain period after the earthquake is more important. It is observed that people are staying nights after nights in playgrounds and open streets not knowing that open streets can be even more dangerous after earthquake. Frantic calls to experts and structural engineers are made to understand what to do and what not to do. The point that is missed is that not much can be done during those emergencies, and expert advices are not given due importance as emotions run high and hence more casualties. The point that is to be taken is that provisions should be made in advance such that the emergencies can be averted or at least minimized through good policy decisions. Good policy decisions can help minimize loss of life and property and more importantly the mental traumas that humanity suffers during and after the event.

One such good policy that we can propose as structural engineer is the **construction of steel buildings** (as proposed in the paper) in highly seismic prone areas in such a way that it is least affected by earthquake.

With this knowledge as back ground, our clear intent is to propose the design of a steel building with structural components (closed hollow steel sections SHS/RHS) and non-structural components (puff panels for walls and steel profiles as floor) such that seismic effect of the buildings can be eliminated or reduced to a great extent by reducing the seismic weight of the building, such that loss of life and property and more importantly mental trauma can be reduced.

The problem is that, human memory is short and we tend to forget everything over a period of time, but responsible authorities should not miss the point.

3. PROBLEMS WITH CONVENTIONAL DESIGN

Basic problems with conventional RCC design are:

- Heavy weight of building and hence high seismic effect.
- Depleting natural resources in the form of fine and coarse aggregates (which are used as raw materials) thus weakening the earth and on the other hand, additional pressure in the form of heavy buildings are put on it.
- Resulting effects are frequent earthquakes, landslides and storm floods.
- High restoration time and cost of affected buildings and hence greater effect on economy.

It is to be noted that due to ease and low cost of construction, RCC building will continue to be used, but at least for selected purposes, buildings as proposed in the paper should be used. This will have desirable effect on economy of the country in longer term.

4. EARTHQUAKE – WEIGHT OF BUILDING – DUCTILITY

It is a well known fact that seismic forces are reduced with reduction of weight of building. That ductile behavior of steel is effective in dissipating seismic forces during the period of motion and comes back to the original position most of the time without much damage. Even if there are damages, it is very limited and easily repairable. The paper uses these well known facts and advantages to design a building with steel hollow sections in earthquake zone IV which is least affected by earthquake forces.

5. BRACINGS – TIME PERIOD – DISPLACEMENT

The building proposed in this paper has been designed with all shear connections. Hence vertical bracings have been used. It has been observed during analysis that placing and quantum of bracings plays a key role in controlling the overall stiffness and hence the time period of the building. Higher quantum of bracings will increase the stiffness and also the earthquake forces which are not desirable. On the other hand inadequate bracings will increase displacement/drift of the building which a steel building will be efficient to resist because of ductility but will cause discomfort to inhabitants. Hence proper judgment is to be used to place bracings. It can vary with configuration of building. Proper review of analysis results will be required before proceeding with design. In the case of low weight building bracing system should not cause any adverse effect. Moreover, it will establish structural stability in the building skeleton frames.

6. DESIGN OF SIX STOREY STEEL BUILDING IN EARTHQUAKE ZONE-IV WITH IMPORTANCE FACTOR 1.5, AS PER IS-1893 CODE:

- Materials used for columns, beams and bracings would be closed steel square/rectangular hollow sections of yield strength fy = 315 N / mm^2.
- Partition walls would be of low weight puff panels or glass as per architectural requirements.
- Flooring would be of stiffened steel plate of 6mm thickness/wooden with horizontal bracings.
- Response spectrum analysis has been done. Cross checked by p-delta analysis.
- Wind analysis has been performed. Basic wind speed assumed as 47m/sec.
- Following drawings are furnished to show the structural arrangement and achieved sections.
- Connections to be provided as per analysis assumptions.
- Ductile property of steel an advantage for earthquake resistant design has been acknowledged.
- Importance has been given to reduction of weight of building by using low weight structural and non structural materials.

Intent is to design a building in earthquake zone IV in such a way that it is least affected by earthquake.

7. RESULTS

Results achieved justify the intent to a great extent and are summarized below:

- Design results reveal that more than 90% of the members are critical in *Dead load, Live load and Wind Load* combinations.
- Even the balance 5-10% of the members which show criticality to earth quake forces are *very marginal*.
- Only those members (mainly columns) which are in proximity to the vertical bracings show criticality to earthquake forces which need slightly heavier sections.

8. CONCLUSIONS

- Findings encourage the initial assumption of encouraging steel buildings in highly seismic prone areas.
- Initial cost can be an issue compared to RCC building.
- Government initiative needs to be taken such that all important buildings such as railway stations, airports, bureaucratic offices, municipal offices, hospitals, telephone exchanges which are run by govt. Should be steel buildings.
- Then it should be extended to all schools, colleges and other buildings which are marked important as per IS-1893.
- Regulations should be in place to encourage such building in **zone V** and **hilly areas in zone IV**.
- Should be made mandatory **beyond a certain height** in zone IV and hilly areas in any zone.

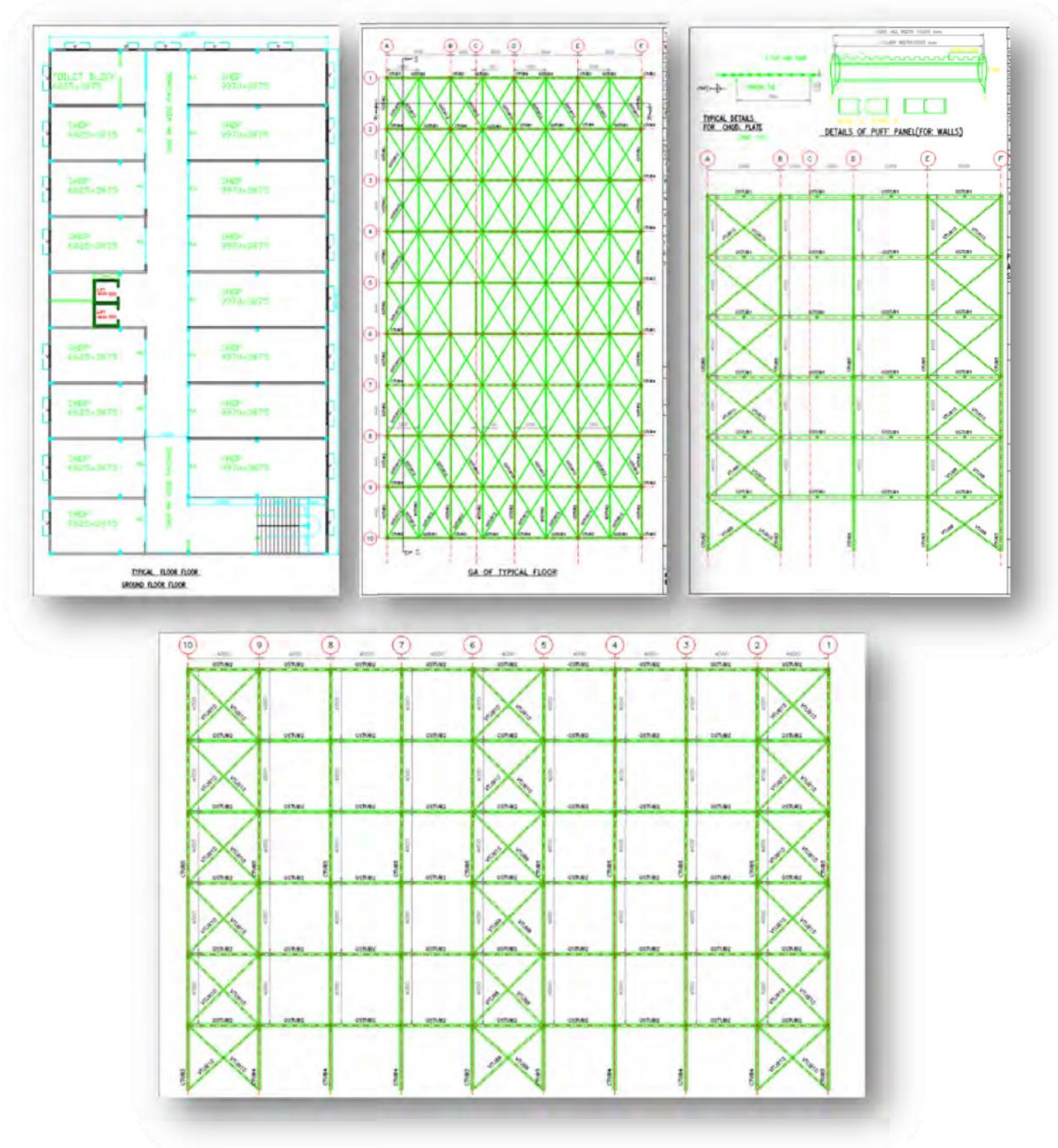

Figure 1: Building Plans and Elevations

N.B.: This work is applied for patent as per IPO India (application no. : 202031040622)

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[2] IS: 1893 (2016) – Indian standard of criteria for earthquake resistant design of structures

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BOLTS- COMPARING CAPACITIES

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1. INTRODUCTION

Connections form an important part of any structure and are designed more conservatively than members. This is because, connections are more complex than members to analyse, and the discrepancy between analysis and actual behaviour is large. Further, in case of overloading, we prefer the failure confined to an individual member rather than in connections, which could affect many members and can cause collapse or global failure of structure.

The type of connection designed has an influence on member design and so must be decided even prior to the design of the structural system and design of members.

Number of Bolts required in a connection depends upon the forces acting on it and the nominal load carrying capacity of bolts. The calculation of nominal load carrying capacity of the bolt depends upon the methodology of calculation adopted by different codes. The basic formula differs. So the capacities of bolts differ from code to code.

This article compares the nominal load carrying capacities of commonly used bolts of property class 4.6, 5.6, 5.8 and 6.8 under the following codes:

1. IS: 800-2007 in the Limit State Method
2. IS: 800-2007 in the Working Stress Method (Chapter-11)
3. IS: 800- 1984 which is in the working stress method

2. TYPES OF BOLTS

Bolts used in steel structures are of three types: 1) Black Bolts, 2) Turned and Fitted Bolts and 3) High Strength Friction Grip (HSFG) Bolts.

The International Standards Organisation designation for bolts, also followed in India, is given by Grade *x.y*. In this nomenclature, *x* indicates one-tenth of the minimum ultimate tensile strength of the bolt in MPa and the second number, *y*, indicates one-tenth of the ratio of the yield stress to ultimate stress, expressed as a percentage. Thus, for example, grade 4.6 bolt will have a minimum ultimate strength 400 MPa and minimum yield strength of 0.6 times 400, which is 240 Mpa.

Black bolts are unfinished and are made of mild steel and are usually of Grade 4.6. Black bolts have adequate strength and ductility when used properly; but while tightening the nut snug tight (“Snug tight” is defined as the tightness that exists when all plies in a joint are in firm contact) will twist off easily if tightened too much. Turned –and- fitted bolts have uniform shanks and are inserted in close tolerance drilled holes and made snug tight by box spanners. The diameter of the hole is about 1.5 to 2.0 mm larger than the bolt diameter for ease in fitting. High strength black bolts (grade 8.8) may also be used in connections in which the bolts are tightened snug fit.

In these *bearing type of connections*, the plates are in firm contact but may slip under loading until the hole surface bears against the bolt .The load transmitted from plate to bolt is therefore by bearing and the bolt is in shear. Under dynamic loads, the nuts are liable to become loose and so these bolts are not allowed for use under such loading. In situations where small slips can cause significant effects as in beam splices, black bolts are not preferred. However, due to the lower cost of the bolt and its installation, black bolts are quite popular in simple structures subjected to static loading.

Turned and fitted bolts are available from grade 4.6 to grade 8.8. For the higher grades there is no definite yield point and so 0.2% proof stress is used.

HSFG bolts are high strength bolts of property class 8.8 and above. This discussion is not considering this type of bolts.

3. FORCE TRANSFER MECHANISM

Force transfer mechanisms of bearing, Shearing and Axial Tension type of bolted connections are described for Black and Turned and fitted bolts. Force transfer mechanism for HSFG bolts are through friction and are not considered in the discussion.

Force transfer by shear in bolts

Fig. 1(*a*) shows the free body diagram of the shear force transfer in bearing type of bolted connection. It is seen that tension in one plate is equilibrated by the bearing stress between the bolt and the hole in the plate. Since there is a clearance between the bolt and the hole in which it is fitted, the bearing stress is mobilised only after the plates slip relative to one another and start bearing on the bolt .The section *x*-*x* in the bolt is critical section for shear. Since it is a lap joint there is only one critical section in shear (single shear) in the bolt .In the case of butt splices there would be two critical sections in the bolt in shear (double shear), corresponding to the two cover plates.

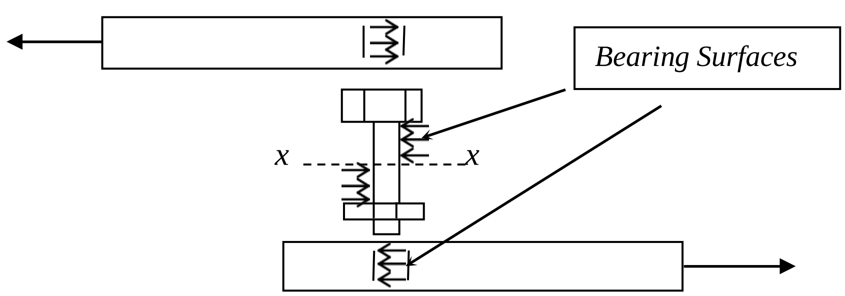

Fig 1(a) Shearing &Bearing Connection

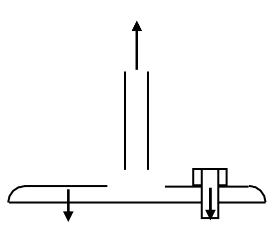

Fig 2(a) Tension type

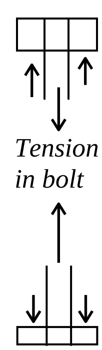

Fig 2(b) Tension in Bolt

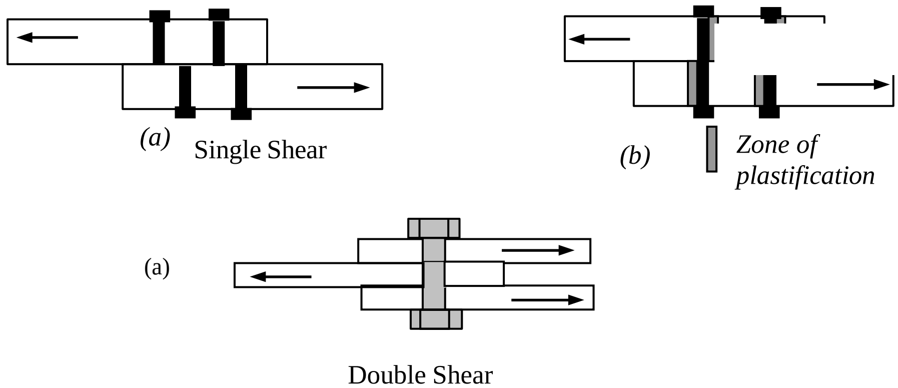

Fig 3 Shear Type Connection, Shearing of bolts

Transfer of tension by bolts

The free body diagram of the tension transfer in a bearing type of bolted connection is shown in Fig. 2(*a*).

In connections made with bearing type of bolts, the behaviour is linear until i) yielding takes place at the net section of the plate under combined tension and flexure or ii) shearing takes place at the bolt shear plane or iii) failure of bolt takes place in bearing, iv) failure of plate takes place in bearing and v) block shear failure occurs. The first three is described below.

4. CALCULATIONS OF CAPACITY OF BOLTS CONFORMING TO DIFFERENT CODES

A. Calculation of Capacity of Bolt as per IS: 800-2007 Limit State Method (LSM)

1. *Shear capacity* :The design strength of the bolt, V_{dsb}, as governed shear strength is given by

$V_{dsb} = V_{nsb} / \gamma_{mb}$

V_{nsb}= nominal shear capacity of a bolt, calculated as follows:

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

f_u = ultimate tensile strength of a bolt

n_n = number of shear planes with threads intercepting the shear plane

n_s = number of shear planes without threads intercepting the shear plane

A_{sb} = nominal plain shank area of the bolt

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread

γ_{mb} =Partial safety factor for bolted connection with bearing type bolts

The underlying assumption behind the design of bolted connections, namely that all bolts carry equal load is not true in some cases as mentioned below.

2. *Bearing strength:* The design bearing strength of a bolt on any plate, V_{dpb}, as governed by bearingis given by

$V_{dpb} = V_{npb} / \gamma_{mb}$

V_{npb} = nominal bearing strength of a bolt, calculated as follows:

$V_{npb} = 2.5\ k_b\ d\ t\ f'_u$

k_b is smaller of $\frac{3}{3d_0}; \frac{p}{3d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0$

e, p = end and pitch distances of the fastener along bearing direction

d_0 = diameter of the hole

f'_u = smaller of f_{ub}, f_u

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively

d = nominal diameter of the bolt

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or, if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking

In the direction normal to the slots in slotted holes the bearing resistance of bolts in holes other than standard clearance holes is reduced by multiplying the bearing resistance obtained (i.e., V_{npb}), by 0.7 for over size & short slotted holes or 0.5 for long slotted holes.

3. *Tensile capacity:* A bolt subjected to a factored tensile force (T_b) shall satisfy

$T_b \leq T_{db}$ *Where,* $T_{db} = T_{nb} / \gamma_{mb}$ and T_{nb} is the nominal tensile capacity of the bolt, given as:
$T_{nb} = 0.90\ f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$

f_{ub} is the ultimate tensile stress of the bolt, f_{yb} is the yield stress of the bolt, A_n is the net tensile stress area. For bolts where the tensile stress area is not defined, A_n is taken as the area at the bottom of the threads and A_{sb} is the shank area of the bolt

B. Calculation of Capacity ofBolt as per IS: 800-2007 Chapter 11 (Working Stress Method)

1. Shear Strength: Actual stress in bolt in shear, f_{sb}, should be less than or equal to permissible stress of the bolt, f_{asb} as given below

The actual stress in bolt in shear, $f_{sb} = V_{sb} / A_{sb}$

The permissible stress in bolt in shear, $f_{asb} = 0.60\ V_{nsb} / A_{sb}$

where V_{sb} = actual shear force under working (service) load

V_{nsb} = nominal shear capacity of the bolt = $f_u (n_n A_{nb} + n_s A_{sh}) / \sqrt{3}$

A_{sb} = nominal plain shank area of the bolt

Nominal capacity of Bolt under single shear $= 0.6 \times (f_u (n_n A_{nb} + n_s A_{sh}) / \sqrt{3})$

2. Actual stress of bolt in bearing on any plate, f_{pb}, should be less than or equal to permissible bearing stress of the bolt/plate, fapb as given below

The actual stress of bolt in bearing on any plate, $f_{pb} = V_{sb} / A_{pb}$

The permissible bearing stress of the bolt/plate, $f_{apb} = 0.60 V_{npb} / A_{pb}$

where V_{npb} = nominal bearing capacity of a bolt on any plate $= 2.5\, k_b\, d\, t\, f_u \times A_{pb}$

A_{pb} = nominal bearing area of the bolt on any plate

Nominal bearing capacity of a bolt on any plate $2.5\, k_b\, d\, t\, f_u \times A_{pb} \times 0.6$

3. Actual tensile stress of the bolt, f_{tb}, should be less than or equal to permissible tensile stress of the bolt, f_{atb}, as given below

The actual tensile stress of the bolt, $f_{tb} = T_s / A_{sb}$

The permissible tensile stress of the bolt, $f_{atb} = 0.60\, T_{nb} / A_{sb}$

where T_s = Tension in bolt under working (service) load

T_{nb} = design tensile capacity of a bolt $= 0.90\, f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$

A_{sb} = nominal plain shank area of the bolt

Nominal Tensile capacity of Bolt $= 0.6 \times 0.90\, f_{ub} A_n < 0.6 \times f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$

C. Calculation of Capacity of Bolt as per IS: 800-1984 (Working Stress Method)

Calculation of Stresses – In calculating shear and bearing stresses in bolt the nominal diameter of the bolt is considered. In calculating the axial tensile stress in a bolt the net area shall be used.

The maximum permissible stress value in a mild steel bolt of property class 4.6 are shown in the table:

Description of Fasteners	Axial Tension σ_{tf}(MPa)	Shear τ_{vf}(MPa)	Bearing σ_{pf}(MPa)
Property Grade 4.6			
Close Tolerance and turned bolts	120	100	300
Bolts in clearing holes	120	80	250

The permissible stress in a bolt (other than HSFG) of property class higher than 4.6 shall be those given in above table multiplied by the ration of its yield stress or 0.2 % proof stress or 0.7 times tensile strength which ever is lesser, to 235 MPa.

Hence the maximum permissible stress for bolts of property grade 5.6, 5.8, 6.8 are calculated which are as follows:

Description of Fasteners	Axial Tension σ_{tf} (MPa)	Shear τ_{vf} (MPa)	Bearing σ_{pf} (MPa)
Property Grade 5.6			
Close Tolerance and turned bolts	153	128	383
Bolts in clearing holes	153	102	320
Property Grade 5.8			
Close Tolerance and turned bolts	179	149	447
Bolts in clearing holes	179	119	372
Property Grade 6.8			
Close Tolerance and turned bolts	214	179	536
Bolts in clearing holes	214	143	447

The calculated bearing stress of a bolt on the parts connected by it shall not exceed

(a) the value of f_y for bolts in clearance holes

(b) the value 1.2 f_y for closed tolerance and turned bolts. f_y is the yield stress of the connected parts.

The basic formulas used in finding out the capacities of the commonly used sizes of bolts under property class 4.4, 5.6, 5.8, 6.8 under Shear, Bearing, axial tension are as follows:

	IS:800-1984	IS:800-2007 LSM	IS:800-2007 WSM
Tension	$\sigma_{tf} x A_n$	$0.90 f_{ub} A_n / \gamma_{mb}$	$0.90 f_{ub} A_n x 0.6$
Shear	$\tau_{vf} x A_{sh}$	$(f_u (n_n x A_{nb} + n_s x A_{sb})/\sqrt{3})/ \gamma_{mb}$	$(f_u (n_n x A_{nb} + n_s x A_{sb})/\sqrt{3}) x 0.6$
Bearing	$\sigma_{pf} x A_{sh}$	$2.5 k_b d t f_u / \gamma_{mb}$	$2.5 k_b d t f_u x 0.6$

Capacities of Bolts are calculated in the Annexure.
Following are the assumptions:

1. The capacity calculation under shear is for single shear
2. The capacity calculation for bearing is for connecting a single M.S. plate of thickness 10 mm.

5. OBSERVATION AND CONCLUSION

It is observed that the Load carrying capacity calculated considering IS 800-2007, Limit State Method under shearing, bearing and axial tension has the maximum capacity followed by calculation conforming to IS 800-2007 working stress method and then by IS 800-1984, working stress method.

Thus the calculations of load carrying capacity by IS:800-2007 gives most economic solution for connection design.

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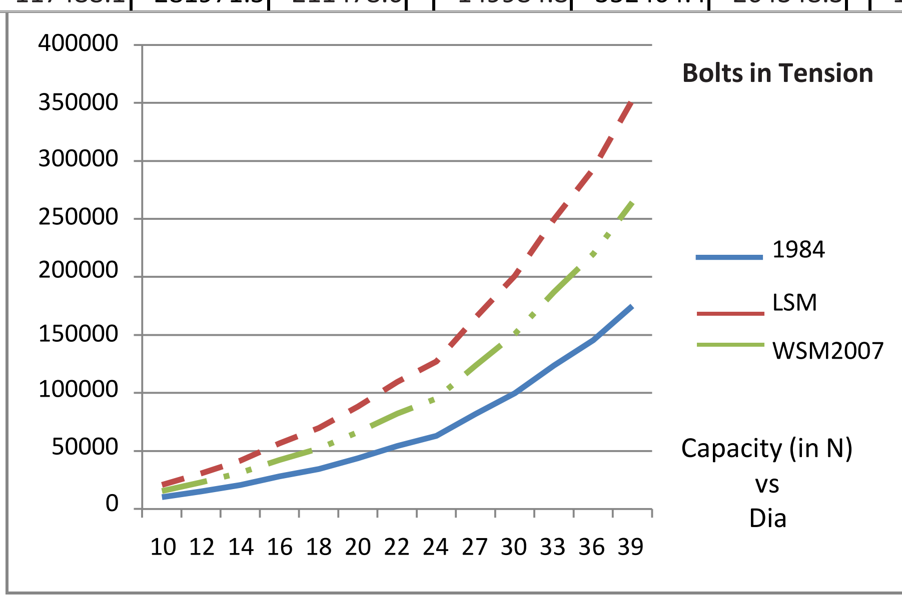
IS: 800-2007, IS:800-1984, IS: 1363- 2002, IS:1367- 2002.

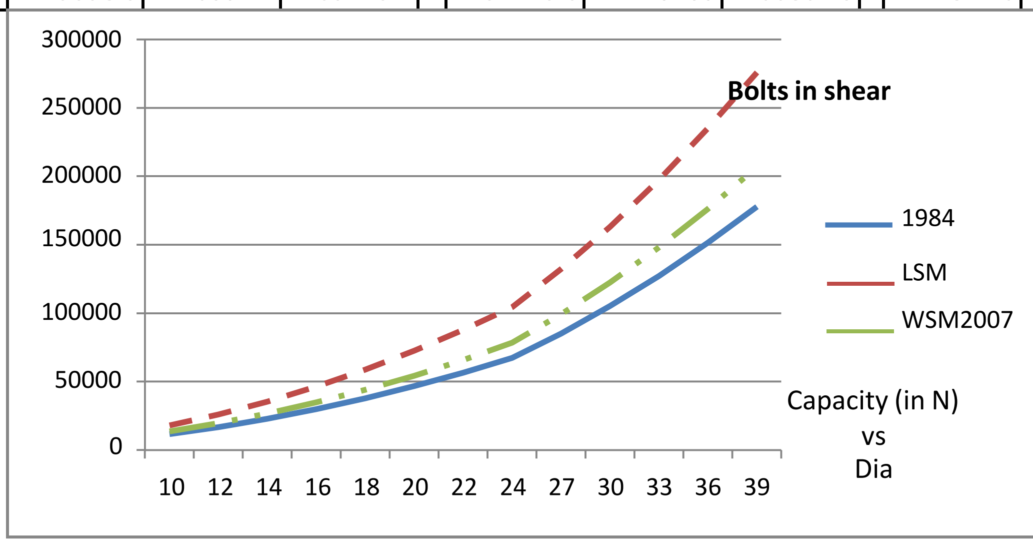
TENSION CAPACITY OF BOLTS IN NEWTON (N)

			Grade 4.6			Grade 5.6			Grade 5.8		
Bolt	Nom.Shank	Net area	**Tension Capacity (N)**			**Tension Capacity (N)**			**Tension Capacity (N)**		
Dia	Area		Ultimate	400		Ultimate	500		Ultimate	500	
			Yield	240		Yield	300		Yield	400	
mm	(mm^2)	(mm^2)	1984	LSM	WSM2007	1984	LSM	WSM2007	1984	LSM	WSM2007
10	78.5	57.305	6876.6	16503.84	12377.88	8778.638	20629.8	15472.35	10241.74	20629.8	15472.35
12	113.04	84.78	10173.6	24416.64	18312.48	12987.57	30520.8	22890.6	15152.17	30520.8	22890.6
14	153.86	115.395	13847.4	33233.76	24925.32	17677.53	41542.2	31156.65	20623.79	41542.2	31156.65
16	200.96	156.7488	18809.86	45143.65	33857.74	24012.58	56429.57	42322.18	28014.68	56429.57	42322.176
18	254.34	193.2984	23195.81	55669.94	41752.45	29611.67	69587.42	52190.57	34546.95	69587.42	52190.568
20	314	244.92	29390.4	70536.96	52902.72	37519.66	88171.2	66128.4	43772.94	88171.2	66128.4
22	379.94	303.952	36474.24	87538.18	65653.63	46562.86	109422.7	82067.04	54323.34	109422.7	82067.04
24	452.16	352.6848	42322.18	101573.2	76179.92	54028.31	126966.5	95224.9	63033.03	126966.5	95224.896
27	572.265	457.812	54937.44	131849.9	98887.39	70132.9	164812.3	123609.2	81821.72	164812.3	123609.24
30	706.5	558.135	66976.2	160742.9	120557.2	85501.53	200928.6	150696.5	99751.79	200928.6	150696.45
33	854.865	692.4407	83092.88	199422.9	149567.2	106076	249278.6	186959	123755.4	249278.6	186958.98
36	1017.36	813.888	97666.56	234399.7	175799.8	124680.7	292999.7	219749.8	145460.8	292999.7	219749.76
39	1193.985	979.0677	117488.1	281971.5	211478.6	149984.8	352464.4	264348.3	174982.3	352464.4	264348.28



SHEAR CAPACITY OF BOLTS IN NEWTON (N)

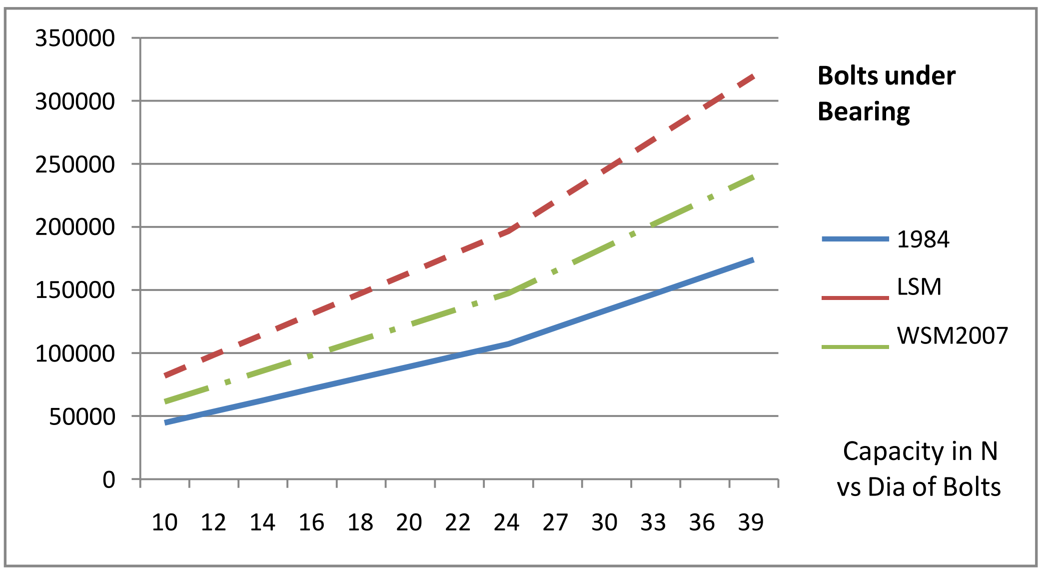
Bolt	Nom.Shanl	Net area	Grade 4.6 **Shear Capacity (N)**			Grade 5.6 **Shear Capacity (N)**			Grade 5.8 **Shear Capacity (N)**		
	Area		Ultimate	400		Ultimate	500		Ultimate	500	
mm	(mm^2)	(mm^2)	Yield	240		Yield	300		Yield	400	
			1984	LSM	WSM2007	1984	LSM	WSM2007	1984	LSM	WSM2007
10	78.5	57.305	7850	14503.04	10877.28	10021.28	18128.8	13596.6	11691.49	18128.8	13596.599
12	113.04	84.78	11304	20884.38	15663.28	14430.64	26105.47	19579.1	16835.74	26105.47	19579.102
14	153.86	115.395	15386	28425.96	21319.47	19641.7	35532.44	26649.33	22915.32	35532.44	26649.334
16	200.96	156.7488	20096	37127.78	27845.83	25654.47	46409.72	34807.29	29930.21	46409.72	34807.293
18	254.34	193.2984	25434	46989.85	35242.38	32468.94	58737.31	44052.98	37880.43	58737.31	44052.98
20	314	244.92	31400	58012.16	43509.12	40085.11	72515.19	54386.4	46765.96	72515.19	54386.395
22	379.94	303.952	37994	70194.71	52646.03	48502.98	87743.38	65807.54	56586.81	87743.38	65807.538
24	452.16	352.6848	45216	83537.5	62653.13	57722.55	104421.9	78316.41	67342.98	104421.9	78316.409
27	572.265	457.812	57226.5	105727.2	79295.36	73055.11	132158.9	99119.21	85230.96	132158.9	99119.206
30	706.5	558.135	70650	130527.3	97895.51	90191.49	163159.2	122369.4	105223.4	163159.2	122369.39
33	854.865	692.4407	85486.5	157938.1	118453.6	109131.7	197422.6	148067	127320.3	197422.6	148066.96
36	1017.36	813.888	101736	187959.4	140969.5	129875.7	234949.2	176211.9	151521.7	234949.2	176211.92
39	1193.985	979.0677	119398.5	220591.2	165443.4	152423.6	275739	206804.3	177827.6	275739	206804.27



BEARING CAPACITY OF BOLTS IN NEWTON (N)

Grade 4.6 Grade 5.6 Grade 5.8

Bolt	Nom.Shank	Net area	Bearing Capacity (N)			Bearing Capacity (N)			Bearing Capacity (N)		
	Area		Ulti. Bolt	400		Ulti. Bolt	500		Ulti. Bolt	500	
			Yield	240		Yield	300		Yield	400	
			Ult Plate	410		Ult Plate	410		Ult Plate	410	
mm	(mm^2)	(mm^2)	1984	LSM	WSM2007	1984	LSM	WSM2007	1984	LSM	WSM2007
10	78.5	57.305	30000	82000	61500	38297.87	82000	61500	44680.85	82000	61500
12	113.04	84.78	36000	98400	73800	45957.45	98400	73800	53617.02	98400	73800
14	153.86	115.395	42000	114800	86100	53617.02	114800	86100	62553.19	114800	86100
16	200.96	156.7488	48000	131200	98400	61276.6	131200	98400	71489.36	131200	98400
18	254.34	193.2984	54000	147600	110700	68936.17	147600	110700	80425.53	147600	110700
20	314	244.92	60000	164000	123000	76595.74	164000	123000	89361.7	164000	123000
22	379.94	303.952	66000	180400	135300	84255.32	180400	135300	98297.87	180400	135300
24	452.16	352.6848	72000	196800	147600	91914.89	196800	147600	107234	196800	147600
27	572.265	457.812	81000	221400	166050	103404.3	221400	166050	120638.3	221400	166050
30	706.5	558.135	90000	246000	184500	114893.6	246000	184500	134042.6	246000	184500
33	854.865	692.4407	99000	270600	202950	126383	270600	202950	147446.8	270600	202950
36	1017.36	813.888	108000	295200	221400	137872.3	295200	221400	160851.1	295200	221400
39	1193.985	979.0677	117000	319800	239850	149361.7	319800	239850	174255.3	319800	239850



DURABILITY ASPECTS OF THE COLD-FORMED STEEL STRUCTURES

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ABSTRACT

Usage of cold-formed steel structural components, which consists of slender plate elements, in the construction of residential and industrial buildings is on the rise. The performance and design of such components are being adequately addressed in many codes of practices, such as AISI-S200, Eurocode-3, AUS-NZ and IS:801. However, the durability of CFS components are not adequately covered. As these elements are thin, the life of the CFS structures reduce drastically, if they are exposed to corrosive environments and left to corrode without the protection. In view of this, different studies, guidelines and protection methods are prescribed in codes of practices. This paper reviews the guidelines adopted in various codes of practices and literature. Based on the review, it is found that metallic coatings, such as galvanizing and galvalume, are generally adopted in thin-walled components. Galvalume coatings are applied at the manufacturing stage itself as a pre-coated sheets/coils. Thickness of the metallic coatings are adopted by the manufacturers as per the standards as well as to the customer's specifications, based on the environmental classifications or exposure conditions.

Keywords: galvanizing, galvalume, grade of coating, Zn-Al alloy, exposure condition, white chalking, sacrificial anode.

1. INTRODUCTION

Cold-formed steel structural components consists of slender plate elements which are generally thin, ranging from 0.5-3 mm. These sections are formed through roll mills and press braking methods. Since, these elements are thin, thickness of the members reduce drastically due to corrosion depending on the exposure conditions. In general, the CFS sections are adopted as different load carrying elements: 1) purlins, 2) portal frames 3) solar panel mounting frames, 4) crash barriers, 5) stud-wall frames, 6) roofing and wall cladding sheets, decking sheets etc. as shown in Fig. 1.

Purlins in the steel framing

CFS members in solar mounting frame

Wall cladding

Metal decking for slabs

Fig. 1 Applications of CFS components

In general, these structural elements are provided with metallic coatings through galvanization process or pre-coated in the manufacturing stage itself as sheets/coils.Different grade of coatings is adopted depending on the exposure condition. This paper reviews the background about corrosion process, coating methodologies and the guidelines available in codes of practices.

2. CORROSION MECHANISM

Steel is a versatile construction material that undergoes corrosion through reacting with oxygen and moisture in the air. Iron oxide is the pure and stable form. Steel corrosion is an electrochemical process. The difference in electrode potential cause the negative electrons to flow from the anode to the cathode leaving the positively charged iron atoms. These iron atoms react with OH_ ions in the electrolyte to form iron oxide on the anode and on the cathode region negatively charged ions react with hydrogen ions to form hydrogen gas. Thus corrosion of structural steel is an electrochemical process, in which presence of moisture and oxygen is must. Though steel is solid and painted, water molecules enters into through microscopic pits and cracks. The dissolved oxygen reacts with steel and rust is formed. Further, the iron oxide expands and occupy more space, nearly about six times the original volume. Fig. 2 shows the pictorial representation of corrosion mechanism. There are different forms of corrosion, namely, galvanic corrosion, pitting corrosion, crevice corrosion, bi-metallic corrosion. Rate of corrosion depends on two prime factors: 1) time of wetness and 2) composition of atmospheric air. In atmospheric air, two main contaminants, sulphates and chlorides influence the corrosion rates. In the industrial environment, high concentration of Sulphur exists. It reacts with moisture or water and from Sulphur compounds. Whereas, chloride concentration is very high in marine environments. Both of these impurities increase the rate of corrosion. Hence, based on the concentration of chlorides and sulphates, most of the standards classify the environment based on their concentrations.

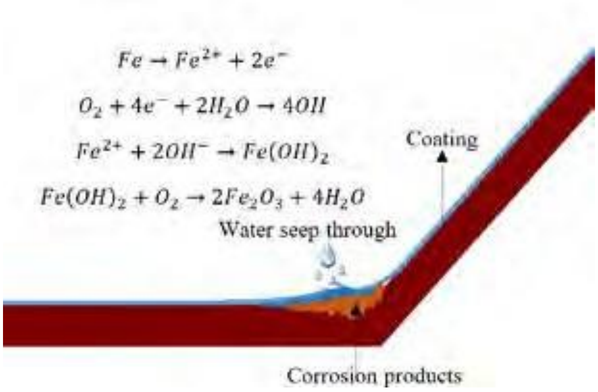

Fig. 2 Mechanism of corrosion in coated steel

Due to the above facts, thickness of the steel is lost and it affects the durability and design aspects of the structures. Most of the protective measures confine to protective coatings and detailing of the members and joints, such that collection of water and accumulation of moisture retaining products, such as dusts and debris. Provision also should be made to drain the water adequately and thus time of wetness can be reduced. Provision of drain holes within the members and connections are suggested. Further, it is also suggested to practice the periodical maintenance including application protective coatings.

3. METALLIC COATINGS

Hot-dip galvanizing and thermal spraying are the commonly adopted methods for protective coating in structural steel components as a pre-coated steel sheets prior to the roll-forming of the desired sections. The methods, such as, electroplating and sherardizing are adopted for fittings, fasteners. Hot-dip galvanizing involves sequential processes. In this process, the steel components are immersed in to the molten zinc having melting point about 450C. Commonly, zinc is used for galvanizing. Zinc and its compounds adhere to the steel substrate and it deforms without any damages to the coating while roll-forming of the sections. Further, zinc-aluminium coatings.

1. Galvanized: pure zinc with 0.2% aluminium is added to form a thin, inhibiting, iron-aluminium.
2. Galfan: 95% zinc, 5% aluminium – known for improved corrosion resistance than galvanized
3. Galvalume: 55% aluminium, 1.5% silicon and 43.5 Zinc alloy – provides superior corrosion resistance

3.1 Galvanization

It is the process of immersing the steel components into the molten zinc bath at about 450 C to develop a protective zinc coating (SFA, 2004). It shall be carried out in accordance with IS: 2629 and the Zinc shall confirm to the grades specified in IS: 209 or IS: 13229.Further, the mass of coating shall be as pet IS 1573 and IS 4759.CFS is galvanized by unwinding the coils and feed them continuously. The Fig. 3describes the galvanizing process, in general. Zinc coating protects the steel surface in two ways. First, it acts as barrier between the environment and steel base. It also protects steel through galvanic or sacrificially at cut edges and scratches. (SCI_P262). Based on the survey and case studies of the light steel framing built using galvanized light steel. The adhered zinc with the steel substrate deforms without cracking and also without detaching itself with the steel surface during the forming process of CFS sections. Hence, galvanizing has become a standard method of protection for CFS sections. It has been proved that the galvanized sections with the standard zinc coating thickness exhibited satisfactory performance within the building envelope.

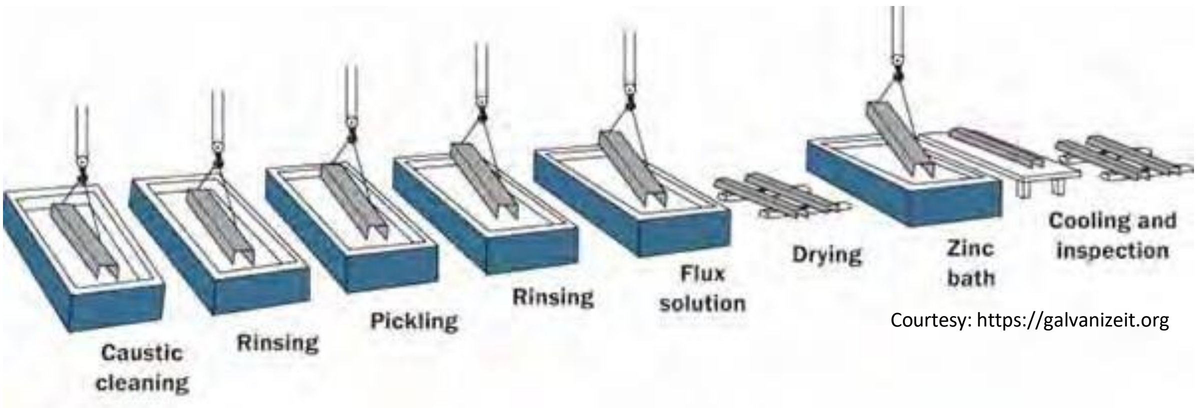

Fig. 3 process of hot-dip galvanizing

The developed coating is the multi-layered zinc-iron compounds. Unlike the other surface coatings, these compounds are integral part of the steel. The photomicrograph of a galvanized coating is shown in Fig.4. The intermetallic layers offer excellent abrasion resistance, as they are harder than steel and strongly adhered to steel surface. The zinc is anodic to steel and hence if the intermetallic layer damages, the adjacent zinc sacrificially protects the exposed steel till the surrounding zinc coating vanish. Different methods, such as coating steel with oil, grease, tar, asphalt, polymer coatings or paints and other corrosion resistant materials.

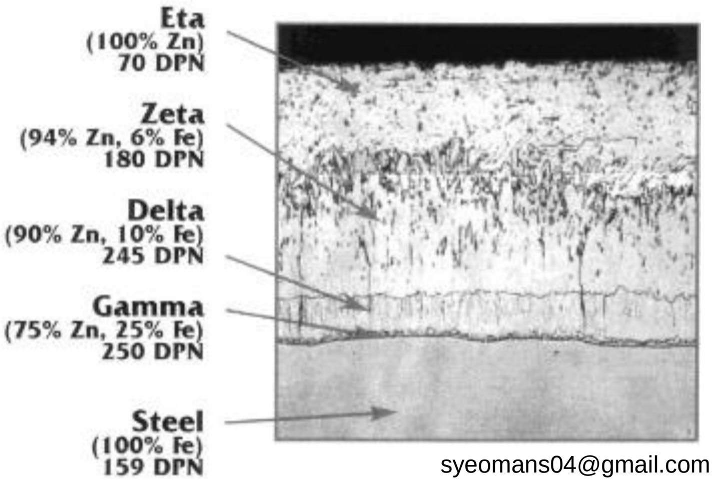

Fig. 4 Photomicrograph of a galvanized coating

4. PERFORMANCE OF METALLIC COATINGS

Corrosion process in metallic coated surfaces (Steel framing alliance- Corrosion protection for life)

The durability or life of metallic coating is dependent on the total continued exposure time of wetness and the atmospheric conditions. The zinc reacts with the atmosphere and as result corrosion products such as zinc oxide, zinc hydroxide and zinc carbonate. The galvanized elements, which are exposed to dry atmosphere and shorter time of wetness/moisture have longer service life. Of these, three products, Zinc carbonate is thin, hard and stable layer. This provides protection to the underlying zinc by preventing further ingress of moisture and atmospheric air. However, these products are partially soluble in water in the presence of SO_2formation of soluble zinc sulphate, which is washed away with running water conditions leading to reduction of zinc coating thickness. Formation of white rust (zinc hydroxide) over the large area is not the indication of serious degradation of zinc coating and it acts protective layer. However, heavy deposits of white rust cannot be ignored, if the surface is exposed to severe environment along with presence of moisture.

Fig. 5 Performance of metallic coating and paint (SFA 2004)

SFA (2004) highlighted the importance of galvanizing and how the cut/scratch of steel coating is protected as in Fig.5. The steel is cathodically protected by the sacrificial corrosion of zinc coating adjacent to the exposed steel surface. This occurs because zinc is more reactive than steel in the galvanic series. In view of this, undercut or scratches in the coating cannot corrode the underlying steel.

Ahmet Gulec et al. (2011) conducted studies on the steel specimens coated with Zn, Zn-Al and Al coatings under accelerated corrosion condition through salt spray test. It was found that the corrosion resistance of Al and Zn-Al coated specimens were found to perform better than Zn coated specimens and Chloride environment. Zn and its alloy coated specimens were protected through sacrificial action and Al coated specimens were protected through its barrier action (SFA, 2004). It was recommended to adopt Zn and Zn-Al alloy coatings for economical and effective corrosion protection of CFS members (Ahmet, 2011 and SFA-2004). Yan Li (2001) assessed the performance of Zn, Zn–25Al alloy and Zn–55Al–Si coated steel wires in seawater. Typical view of the corrosion loss for various coatings are shown in Fig.6. It was found that hot dip Zn–25Al coating exhibit excellent barrier protection over galvanized zinc and Zn–55Al–Sicoating was suffered due to pitting corrosion under sea water. Lawson et al. (2010) also emphasized similar requirements for achieving the durability of the CFS buildings and discussed about it with case studies.

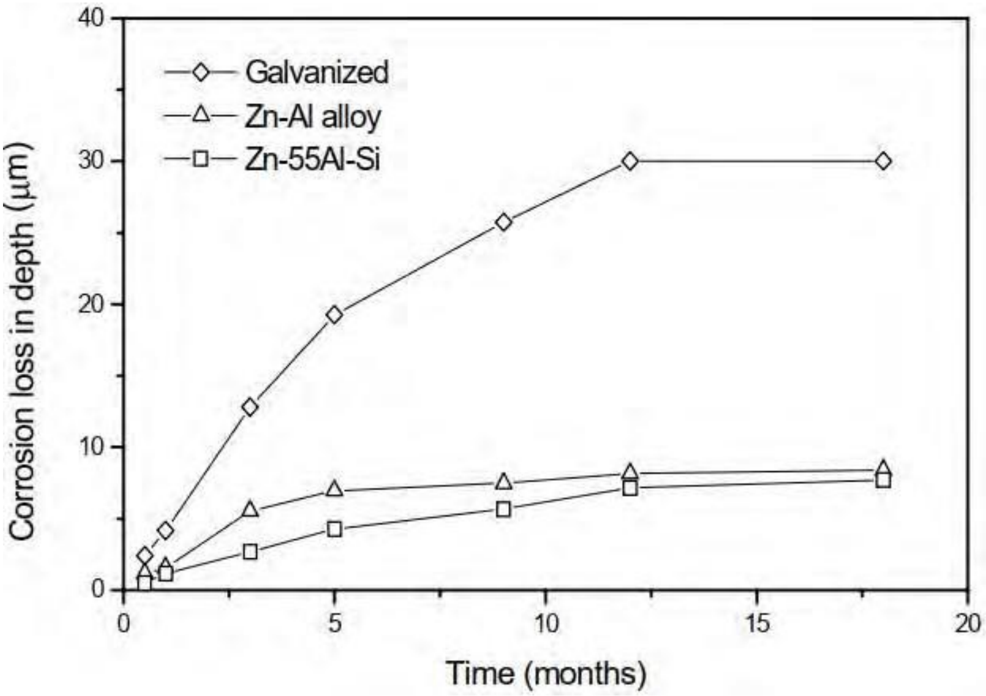

Fig. 6 Corrosion loss as a function of time for hot dip coatings in tidal zone [Ref:20]

5. DURABILITY OF COLD FORMED STEEL COMPONENTS

The duty of the designer/user to define the exposure condition of the building, which will be exposed to the life time, accordingly the required and durable coating should be selected towards protecting the CFS members from undergoing corrosion. Durability of CFS components is depends on the durability of the coating itself. It is as simple as that of how long the coating exists on the given exposure condition. Further, it also depends on the frequency of replacement of the coating or maintenance of it. Hence, to help in the maintenance programme, based on the time to the first major maintenance for corrosion protection, durability shall be expressed under four different classes as follows in accordance with ISO 12944-1&5.

Low (L) up to 7 years

Medium (M) 7 to < 15 years

High (H) 15 to < 25 years

Very high (VH) ≥ 25 years

Periodic maintenance shall be carried out for fading, chalking, contamination, wear and tear and any other deterioration in between major maintenance intervals. The above durability criteria are arrived based on the environmental classification, which classifies the exposure condition in terms of time of wetness and impurities in the atmosphere.

6. ENVIRONMENTAL CLASSIFICATION

As mentioned in the previous sections, the life of coating or life of steel components are dependent on the environmental conditions and their time of wetness. Hence, IS: 14191 classify the corrosivity of the atmosphere is into five categories as in Table 1.

Table 1 Classification of Environmental/ exposure conditions

Category	Corrosivity	Exterior	Interior
C1	Very low	-	Heated buildings with clean atmospheres (offices, shops, schools, hotels etc.)
C2	Low	Atmospheres with low level of pollution. Mostly rural areas.	Unheated buildings where condensation may occur (depots, sports halls)
C3	Medium	Urban and industrial atmospheres, moderate sulphur dioxide pollution. Coastal areas with low salinity.	Production rooms with high humidity and some air pollution, e.g. food-processing plants, laundries, breweries, dairies.
C4	High	Industrial areas and coastal areas with moderate salinity.	Chemical plants, swimming pools, coastal ship- and boatyards.
C5	Very high	Industrial areas with high humidity and aggressive atmosphere Coastal and offshore areas with high salinity.	Buildings or areas with almost permanent condensation and with high pollution.

Further, for the members and components that are located inside the buildings are dependent on the prevailing exposure conditions inside the buildings. To assist in classifying the exposure condition of the indoor conditions, Tabe 2 gives guidance for it. As per ISO 9226:2012, a standard size of specimens, 100 mm x 150 mm x 1mm are exposed to the chosen environment. The sample should be free from oil and grease. The steel specimens with rust stains should be polished with 120 grit paper to remove the visible corrosion products. Three samples should be exposed for one-year duration at the beginning of the chosen environment. After the exposure, the corrosion products shall be removed as per ISO 8407.The corrosivity of indoor can be classified in to five categories (ISO 11844-1,2), IC1 to IC5 as in Table 2.

Table 2 Classification of corrosivity of indoor atmospheres

Category	Corrosivity	Details of structures	Steel, $g/(m^2.a)$	Zinc, $g/(m^2.a)$
IC1	Very low	**Heated spaces** with related humidity below 40% without condensation, i.e. controlled environment (Computer rooms, museums) **Unheated spaces** with dehumidification with no specific pollutants (Military stores for equipment)	≤ 0.07	≤ 0.05
IC2	Low	Relative humidity <50% with fluctuations without condensation (museums and control rooms)	0.071-1.0	0.051-0.25
IC3	Medium	**Heated spaces** with fluctuations in temperature and humidity. (Switch boards in the power industry) **Unheated spaces** with humidity in the range 50-70%, fluctuations in temperature and humidity without risk of condensation (Outdoor telecommunication boxes in rural areas	1.0-10	0.251-0.7
IC4	High	**Heated spaces:** fluctuations in temperature and humidity with elevated levels of pollutants (Electrical service rooms in industrial plants) **Unheated Spaces:** relative humidity more than 70% with low risk of condensation (Outdoor boxes for telecommunications in polluted areas)	10-70	0.701-2.5
IC5	Very high	**Heated spaces:** higher levels of pollutants (H_2S) (Ex: cross connection rooms without efficient pollution control) **Unheated spaces:** High relative humidity and risk of condensation medium and higher levels of pollutants (Storage rooms in basement in polluted areas)	70-200	2.501-5.0

Rate of corrosion is primarily dependant on the time of wetness and deposition rates of sulphur and chloride. Hence, the severity of the impurities and wetness are classified in an ascending order as shown in Table. 3. Further, based on these conditions, the corrosivity of the given atmosphere can be classified based on time of wetness, amount of sulphur, chloride and prosperous as shown in Table 3 and 4(based on IS:14191)

Table 3 Classification of deposition rate of sulphur, chloride and time of wetness

SO_2 (mg/(m^2.d)	Cl (mg/(m^2.d)	Category (hours/annum)
P_0 (≤12)	S_0 (≤3)	τ_1 (≤10)
P_1(13-40)	S_1(4-60)	τ_2 (11-250)
P_2(41-90)	S_2(61-300)	τ_3(250-2500)
P_3(91-250)	S_3(301-1500)	τ_4(2500-5500)
-	-	τ_5(> 5500)

Table 4 Corrosivity category combined impurities in the atmosphere

Wet	Time of wetness														
SO_2 ↓	τ_1			τ_2			τ_3			τ_4			τ_5		
	Chloride (Cl) deposition rate														
	S_0- S_1	S_2	S_3	S_0- S_1	S_2	S_3	S_0- S_1	S_2	S_3	S_0- S	S_2	S_3	S_0- S_1	S_2	S_3
P_0- P_1	C1	C1	C1/2	C1	C2	C3/4	C2/3	C3/4	C4	C3	C4	C5	C3/4	C5	C5
P_2	C1	C1	C1/2	C1/2	C2/3	C3/4	C3/4	C3/4	C4/5	C4	C4	C5	C4/5	C5	C5
P_3	C1/2	C1/2	C2	C2	C3	C4	C4	C4/5	C5	C5	C5	C5	C5	C5	C5
Typically C1/2: C1 or C2															

7. RATE OF CORROSION OF METALS

Values are derived for commonly used metals in different corrosivity category are shown in Tables 5 as per IS: 14191. Further, from Table 5, the C5 category address the members and structures located in the land based structures. Further, to address the corrosion of the members and structures in the offshore conditions, ISO 9223 and ISO 12944-2 has a corrosivity category “CX”, which address the extreme corrosivity category. The rate of corrosion for Carbon steel, zinc and aluminium are shown in Table 5. While Rate of corrosion of carbon steel is very high, it is descending order for zinc and aluminium for a chosen corrosivity category. Further, as the durability class defines the life of coatings before the first major maintenance, Table 6 Shows the guiding corrosion rate of metals of first 10 years of service. According to the values of the first 10 years of corrosion rate, the durability class shall be adopted.

Table 5 Corrosion rate of metals for different corrosivity category

Corrosivity Category	Carbon Steel, g/(m^2.a)	Zinc, g/(m^2.a)	Aluminium, g/(m^2.a)
C1	≤ 10 (1.3)	≤ 0.7 (0.1)	-
C2	11-200 (1.3-25)	0.8-5 (0.2-0.7)	≤ 0.6
C3	201-400 (25-50)	6-15 (0.8-2.1)	0.7-2
C4	401-650 (51-80)	16-30 (2.2-4.2)	3-5
C5	651-1500 (80-200)	31-60 (4.2-8.4)	5-10
CX	1501-5500 (201-700)	61-180 (8.4-25)	>10
C5-I (ISO 12944-2)	>650 to 1500 (80-200)	31-60 (4.2-8.4)	-
C5-M (ISO 12944-2)	>650 to 1500 (80-200)	31-60 (4.2-8.4)	-

Table 6 IS 14321-1995 Guiding corrosion rates for first 10 years

Corrosivity Category	Carbon Steel, (μm/a)	Zinc, (μm/a)	Aluminium, (μm/a)
C1	≤ 0.5	≤ 0.1	≤ 0.01
C2	0.6-5	0.2-2.0	≤ 0.025
C3	6-12	3-8.0	0.026-0.2
C4	13-30	9.0-15.0	-
C5	31-100	16-80	-
CX	>100	>80	-

8. COATING REQUIREMENTS

CFS sections are formed by roll-forming the pre-coated steel coils at room temperature. It is reported that during the roll-forming process, the zinc undergoes elongation resulting in slight reduction in thickness. However, the bond between the zinc coating and underlying steel surface is intact. The durability of coating is dependent on primarily the time of wetness and exposure condition only. It is also reported that the corrosion rate of the zinc in the indoor is 10 times slower than the outdoor conditions. ASTM A1003/A1003M-15 suggested to paint the surfaces as the diffusion of iron into the zinc coating during the galvanizing lead to develop rust stains on the zinc coating. Further, in general during the coating process, zinc-iron coating is normally in dull grey colour due to diffusion of iron in the range 8-12%. For a normal exposure conditions, minimum grade of coating for structural members is 150 g/m^2 and non-structural grade is 120 g/m^2. Heavier coating grade are suggested for more severe exposure conditions, like, coastal areas. In the laboratory evaluation, not more than 10% of coating loss should occur under minimum exposure of 100 and 70 hr duration of salt solution for structural and non-structural members respectively. Loss of coating is indicated by the appearance of red rust.

ASTMA653A653M provides guidelines for zinc-coated (galvanized) or zinc-iron alloy coated by hot dip process sheets (galvannealed) with the coating grade in the range 03-1100g/m^2 for galvanizing and 03-180 g/m^2 for galvannealed coating. Similarly, ASTM A792/A792M stipulates specifications for steel sheets with coating consists of 55% aluminium, 1.6% silicon and balance 43.4% zinc. Coating grades shall be ranging from 100-210 g/m^2. The coated steel sheet should be able to bent through 180° without any flaking of coat on the outside. ASTM A875/A875M provides specifications for steel sheets coated with Zinc-5% aluminium alloy coated by hot-dip process: 45-700 g/m^2. Corrosion rate of steel sheet coated with the above coatings decreases with time due to the formation of passive layer.Non-metallic coatings, such as paintings, are suggested in places where corrosion rate is low, such as dry, low rainfall and low humid areas, interior of buildings (category 1 &2 of annexure-1). The coating shall be of minimum 25 microns, should have 250 hrs of salt solution exposure. Whereas, painted metallic coated samples should have to exposed to 500 hrs of exposure in the salt solution.

9. MINIMUM COATING THICKNESS

The required thickness of the galvanisation is measured by coating weight or by thickness. Based on the guidelines from ASTM standards for various types of galvanizing, AISI (2004) listed out the minimum coating thickness requirements as in Table 7. AISI S200 suggest to adopt the minimum grade of coating as per ASTM A1003 and additional protection measures need not be carried out on the edges, filed cut, drilled or punched holes.

Table 7 ASTM A 1003 Minimum coating requirements

	A653/A653M	A792/A792M	A875/875M
Structural	G60/Z180	AZ50/AZ150	GF30/ZGF90
Non-Structural	G40/Z120	AZ50/AZ150	GF30/ZGF90
G- Galvanizing and AZ – Zinc-Aluminium alloy			

A survey has been made on the various products, such as, cladding sheets, roofing, decking, purlins etc. available in Indian market. It is found that the grade of Zn coating ranging from Z60 to Z600. Since the rate of corrosion of Zinc in galvanized coating is available as given in Table 5, the required grade of Zn coating can be worked out as given in BS EN ISO 14713-1:2009. Currently, the required grade of Zn-Al coating may not be worked out, as the rate of corrosion of it is not available. Hence, it requires further studies with regard to its corrosion rates for various exposure conditions. However, predominantly Zn-Al metallic coting in the range AZ 70-AZ 200, which is used up to C4 exposure category, is provided. Further IS 15961 and IS 15965 recommends the following minimum grade of coatings for the Indian conditions as in Table 8.

Table 8 Recommended grades of Aluminium-zinc alloy (Galvalume) coating

Sl. No.	Durability Class	Atmospheric Classification (IS 14191)	55 Percent Aluminium-zinc Metallic Coating (see Note2)	Typical Top Coat Paint System (see Note1)
(1)	(2)	(3)	(4)	(5)
i)	Class 4	Category C4	AZ 200	Polyester/super durable polyester/polyvinylidene fluoride (PVDF)/water-based-acrylic
ii)	Class 3	Category C3	AZ 150	
iii)	Class 2	Category C2	AZ 150	
iv)	Class 1	Category C1	AZ 70/AZ100	

Notes
1. Different top paint system gives different paint durability at given exposure paint systems with exterior premium durability are for long-term colour and gloss retention requirement. Class 3 denotes products with exterior premium durability compared to Class 2.
2. Refer to IS:15961 and IS:15965

10. COATING REQUIREMENTS FOR C5 AND CX CATEGORY

With regard to the coating requirements for CFS members and structures located in C5 and CX corrosivity category are not provided with any specification with Galvalume coatings. However, the standards SNZ TS-3404, ISO 12944-5 shall be referred for detailed coating requirements for the chosen corrosivity category. Typically, CFS sections and members in C5and CX shall be provided with aluminium-zinc alloy coating having coating thickness of 150 -300 μm or epoxy, polyurethane paint system having thickness in the range 300-500 μm, as per and ISO 12944-5, respectively and may be referred for additional details.Further, the following guidelines are also available for the users.

CFSEI (2012), in their technical note, corrosion of CFS members and structures in the coastal region is covered (Table 9 and Fig 7). It is emphasized that the use of standard coatings may be appropriate for inland buildings. However, for the buildings located in the coastal regions additional care should be taken while using the standard coatings.

Table 9 Classification of exposure condition of building located in coastal area

No.	Area of the building	Description of the Outdoor exposures
1	Partial shelter	Under house storage, covered car parking, column, cantilever elements, components or structures under a covered roof etc. Significant corrosion will occur after 5 to 10 years of exposure.
2	Boldly exposed	Exterior walls with framing. If the elements are dried between the evens of the wetting, corrosion rate will be lower than the partial shelter case.
Description of the enclosed exposures		
3	Vented	The areas which are meant to release the heat and moistures. The areas where the outside air is circulating will be similar to that of the partial shelter. Other areas of the vent will have very low level of corrosion.
4	Unvented	These building components, which are generally filled with foam with air sealant such that outside air flow is prevented, will have less level of corrosion than the above.
5	Interior spaces	In these spaces, the coastal air is sealed and the humidity and temperature will not favour for corrosion to occur. Hence, these areas will have lowest rate of corrosion.

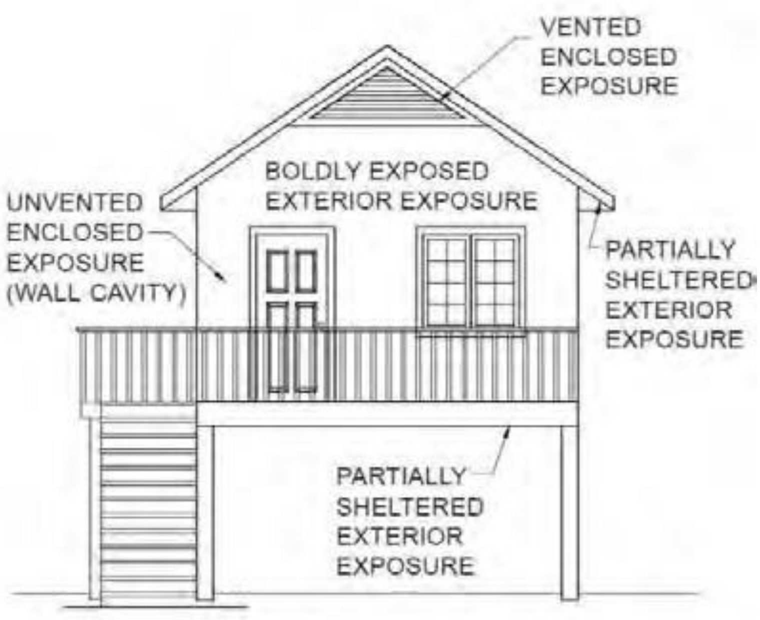

Fig. 7 Different exposure condition of building

One of the ways to address the issues is modifying the boldly exposure condition into a partial exposure condition through 1) covering the exterior 2) vapour retarder/barrier and alternatively sheathing can also be added to the elements. Five classes of buildings are identified, depending on the exposure conditions as shown in Fig. 6. In coastal regions, the provisions should be made for periodic maintenance and thereby damaged regions of the members can be repaired or replaced as in Table 10. The members, which are difficult for repair due to the difficulties in carrying out the periodical maintenance, shall be provided with more corrosion resistant materials or durable coatings.

Table 10 Remedial measures for corrosion protection CFS in coastal areas

Exposure	**Distance from the shoreline**			
	≤ 300 feet	**300-500 feet**	**1500-3000 feet**	**≥3000 feet**
Interior living space	G60 and G40 grade of galvanization for structural and non-structural members. Joints of drywall should be sealed to prevent the air flow.	Coating grades: G40- non-structural G-60: Structural members. In unshielded exposure conditions, taping of joints and sealing the top of wall to prevent air flow.	Coating grades: G40- non-structural G-60: Structural members.	Coating grades: G40- non-structural G-60: Structural members.
Unvented enclosures	G60 grade of galvanizing for sealed walls. For limited airflow walls, minimum G90 grade of galvanizing with without paint on top of it.			Standard G60 grade
Vented enclosures	Use of CFS sections shall be avoided.	Use of CFS sections shall be avoided in unshielded exposure condition.	Use heavier coating near vents or use of pain on	G60 grade

		Use heavier coating near vents or use of pain on galvanizing in other exposure conditions.	galvanizing in on shore wind conditions. G60 grade of galvanizing with periodic inspections	of galvanizing
Bold exteriors and partial shelter	Avoid use of CFS framing	Use of CFS in unshielded exposure conditions shall be avoided Use of CFS sections having minimum G90 grade of galvanizing and periodic maintenance.		
On site storage	Maximum 2 months with precautions	2-4 months	Maximum 4 months	Maximum 6 months

In addition, for CX ISO 12944-9:2018 suggest to adopt the paint coatings as in Table 11. Similar paint coating system specifications for all other corrosivitycategoriesare prescribed in ISO 12944-5:2007. Further, NZS/AS 2312:2002 and SNZ TS 3404:2018 provides similar paint system for all corrosivity and various durability requirements.

Table 11 Minimum requirement for paint systems (ISO 12944-9:2018)

Paint system			Hot-dip galvanizing
CX offshore	Splash and tidel zones of CX	IC4	
primer coat with 3 intermediate coats having minimum 40µm and the top coat with minimum 280 µm.	Zinc rich primer, 3 coat of intermediate 40µm minimum and for other primer minimum 200µm. Top coat should be in the range 450-600 µm.	Primer with one intermediate coat and the final coat of minimum 800µm. If 2 coats of 150 µm is provided top coat shall be minimum 350 µm.	No primer+ 2 coats of zinc rich top coat with thickness of minimum 200µm.

11. MAINTANENCE/PRECAUTIONS TO BE ADOPTED IN METALLIC COATINGS

Metallic coatings require very little maintenance efforts. However, both zinc and zinc-al alloys have their own service life and rate of corrosion in the exposed environment, the following shall be adopted to achieve the desired durability of the coating.

Formation of white rust in galvanized coating

During the initial period of the coating immediately, a chrome is applied to provide short term protection to the zinc coating. Commonly occurring problem in the galvanized surface is formation of bulky, white, powdery deposits, which is generally termed as white rust or wet storage stain. Both galvanized and Galvalume coating contain 100% and 55% zinc. Zinc reacts with water to form zinc hydroxide. This occurs mainly during the storage of materials. In an oxygen deficient environment (as in Fig. 8), where the surface is not getting the required oxygen for formation of stable oxide formation, water reacts with zinc progressively on the products that are stacked tightly for longer period of time.

Fig. 8 Formation of white rust in the Zinc galvanized coated CFS members

This problem can be avoided by keeping the storage dry, packing them by permitting air circulation, allowing proper drainage of water and treatment for the surface to prevent the moisture contact. These white rust formations can be brushed off, if it is lighter. If the rust formation is moderate, such that the appearance of the affected is damaged moderately and it is unacceptable, loss of thickness is more than 5%, aluminium paint can be applied over the surface after cleaning the white rust. If the affected area is appearing to have black appearance showing rusting, generally termed as corrosion etching, the surface should be cleaned and the affected area can be applied with epoxy-rich paint having minimum thickness as 100 microns.

Storage

The recorded corrosion rate for indoor conditions was about 0.1 μm over three year period. However, water leakage, humidity will accelerate the corrosion. Minimum coating grades as specified in ASTM A1003 would be give adequate protection, as exposure to moisture will not be on a regular basis. Additional protection measures are suggested for aggressive environments. Emphasis was given more on to the bottom track of tracks, as they may collect moisture during erection and service life.

Handling & Transportation

Normal handling during erection or transportation and storage will not damage the zinc coating, as it is highly adherent and abrasion resistant. Further, cutting and fastening also will not affect the performance of the coating due to cathodic action of zinc coating. However, precautions should be taken to prevent formation of white rust while storing. The galvanized products should be stored with proper drainage and ventilation such that they dry faster. Further, site and shop fabrications involve welding process, which may be the localised defect, as it volatilizes the zinc. It may not need any precautions if weld area is small, which shall be spot welds. If the damages are wide, in the case of continuous welds, they should be repaired with zinc rich paint or zinc metallizing.

Zinc provides impervious barrier, which prevents the moisture to penetrate and act as electrolyte. This metallic zinc coating has excellent adhesion and abrasion resistance. It protects the steel galvanically. As zinc is more electronegative than steel, it cathodically protects the steel through sacrificial corrosion, if the steel is exposed due to scratch or cut marks on the zinc coating. However, zinc itself is a reactive material and corrodes slowly over the time.

Summary and conclusions

As the use of CFS components in the construction of buildings is on the rise, their durability over the stability issues are important for the user and the builders. In view of the above, a review has been carried out on the guidelines adopted for the corrosion protection of CFS members and structures located in all category of environmental classifications. It is found that metallic coatings, such as zinc galvanizing and zinc-aluminimum (Galvalume) alloy are commonly adopted. Galvalume is primarily adopted in the manufacturing process of the sheet/coil itself as a pre-coated product. The grade of coating is adopted as per the standards in their respective locality or the specifications of the user. The durability of the such coatings affects the durability of CFS buildings. Based on the review, a collective information with regard to the environmental/corrosivity category, rate of corrosion of different metals including their metallic coatings are presented, including for coastal areas. Based on the review, it can be said that with the available technology and standards a durable CFS members or structures can be built for the entire service life.

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